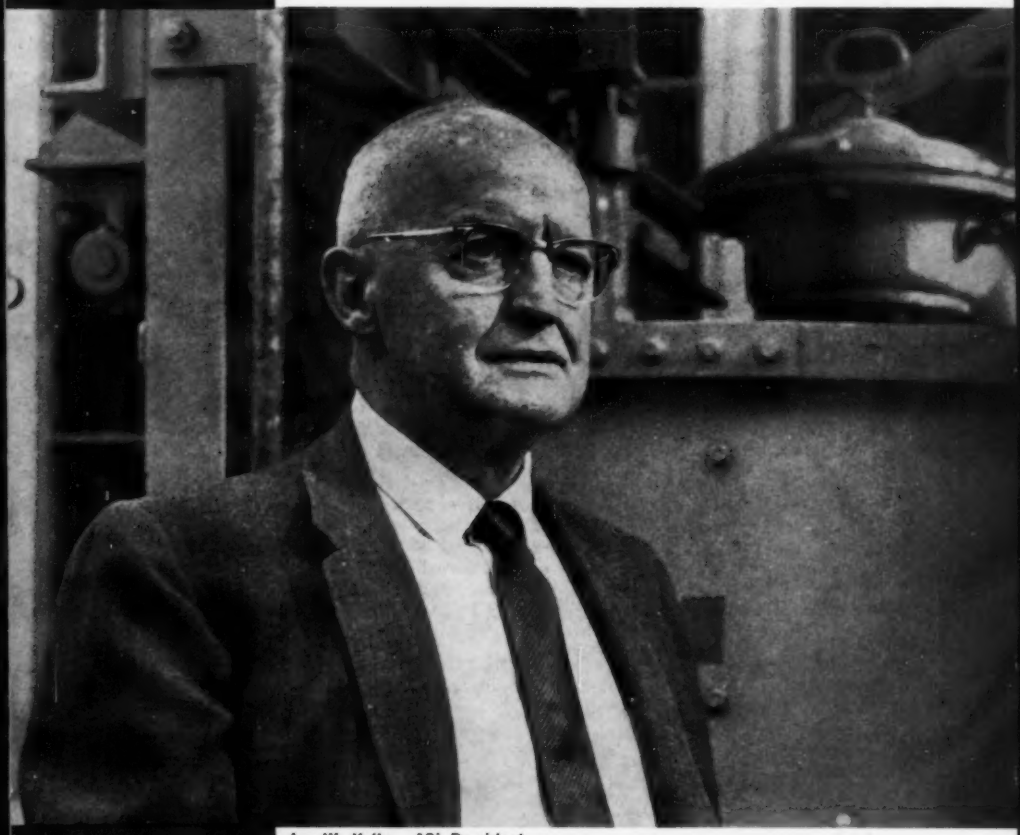
JOURNAL
OF THE AMERICAN CONCRETE INSTITUTE



Joe W. Kelly — ACI President

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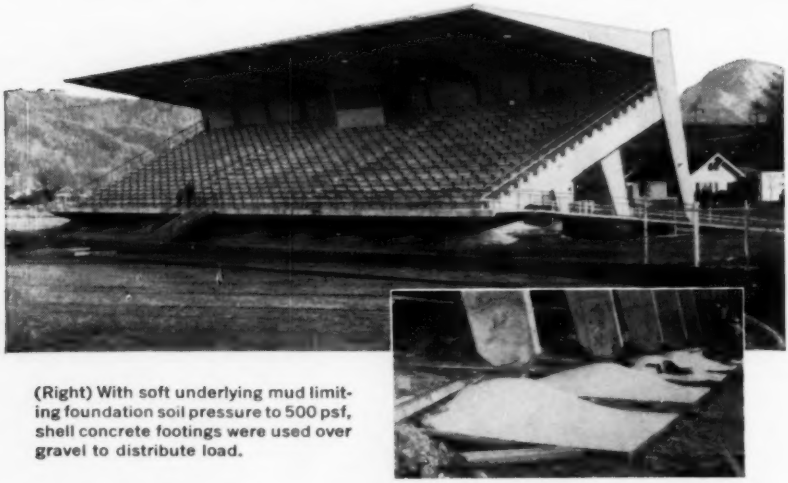

(Right) With soft underlying mud limiting foundation soil pressure to 500 psf, shell concrete footings were used over gravel to distribute load.

● Spartan Stadium is an example of what can be accomplished with modern methods and the most modern of structural materials—prestressed concrete. And once again, Incor®—America's *first* high early strength portland cement—was voted "most valuable player."

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Coming next month in the JOURNAL

EMILIO ROSENBLEUTH and IGNACIO HOLTZ offer a method to simplify computations in determining the "Elastic Analysis of Shear Walls in Tall Buildings." A numerical method is set up to systematize successive cycles of iteration and it is found that extrapolation from the first two cycles supplies almost exact results.

"A Method of Assessing the Probable Fire Endurance of Load-Bearing Reinforced Concrete Columns" is presented by J. H. CLARKE. The methods advocated may be extended to deal with circular columns and other types of reinforcement.

"A Multipurpose Concrete Building of Precast Thin-Shell Panels" is described by ARSHAM AMIRIKIAN. Author covers some of the problems of framing arrangement, design, assembly, and jointing.

ROBERT G. MATHEY and DAVID WATSTEIN reveal results of their investigation to determine the "Effect of Tensile Properties of Reinforcement on the Flexural Characteristics of Beams."

"Carbon Dioxide in Hydrated Portland Cement" by W. F. COLE and B. KROONE is based on a study made of the way in which carbon dioxide is held in carbonated samples of portland cement mortar and of calcium silicate hydrate using differential thermal, thermogravimetric, and x-ray diffraction methods.

Discussion of papers published in the October, November, and December, 1959, issues will appear in the June JOURNAL in accordance with the Institute's quarterly publication schedule for *Proceedings* discussion.

On the Cover — JOE W. KELLY, professor of civil engineering, University of California, Berkeley, was installed as president of the American Concrete Institute for a 1-year term at the March, 1960, annual meeting in New York.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

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Title No. 56-54

Phil M. Ferguson, professor of civil engineering, University of Texas, Austin—retiring president of ACI—points to the expanding role of ACI internationally and the increased activity of technical committees. The industry is reminded of the problems needing solution, three examples being uniformity of concrete strength and properties, adequate inspection, and industry research.

ACI in an Expanding Role

PHIL M. FERGUSON

■ EACH YEAR THE PRESIDENT OF ACI has the privilege of presenting a few personal remarks to the annual convention. It seems appropriate this year that this discussion revolve around (1) the increased role which ACI is playing internationally, (2) the increased activity under our revised rules for committee work, and (3) some of the problems which face ACI.

In 1924 your president made his first trip to New York City to start his first full time engineering job. In 1930 he became a member of ACI. The last 30 years have made fully as many changes in the American Concrete Institute as they have in the New York skyline.

INTERNATIONAL OPERATIONS

In membership, the 2565 members of 1930 have increased to something over 10,200, after dropping to less than 1200 in 1935. The present organization owes a real debt to the officers and staff who persevered during the depression years of the thirties and kept the Institute a going organization.

Membership growth abroad has more than kept pace with that in this country. There are now, outside the United States and Canada, 1800 members in 84 different countries with an additional 1200 overseas subscribers to the ACI JOURNAL in 71 countries, including 12 countries which are not represented by members. The total of members and these subscribers thus becomes about 12,000, of which almost exactly one out of every four are outside the United States and Canada. Thus

ACI is more of an international organization in its influence than many of us have realized. Not only do people of other countries learn of what we are doing here, but we are the recipients of many discussions and papers from these countries. The international flavor in ACI will doubtless increase, rather than decrease, as jet travel brings us closer together.

Past President Douglas McHenry should receive credit for initiating positive steps, during his presidency in 1958, to bring about formal contacts with European engineers. In addition to his personal contacts abroad, he appointed a liaison committee to discuss with CEB, the European Committee on Concrete, proper ways and means of closer cooperation. The first result of this was an invitation for ACI to send observers to the meeting of CEB in Vienna last April. It should be noted that this was a high level conference, official or semi-official, with representatives from most of Free Europe. Its purpose was the pooling of research reports and research efforts as the first move in the direction of a unified basis for building codes in the respective countries. These ACI representatives brought back the most encouraging accounts of the reputation of ACI and its JOURNAL in Europe.

Since that time it has been my privilege to name the three original ACI observers to an official ACI-CEB Collaboration Committee. The ACI representatives on this committee are: Eivind Hognestad of the Portland Cement Association Research and Development Laboratories; Chester P. Siess of the University of Illinois; and Raymond C. Reese, chairman of the ACI Building Code committee and the incoming vice-president of ACI. The permanent European members are Prof. F. Levi of Italy and Prof. G. Wästlund of Sweden, and there is a third European membership which rotates among a group of six men. With cooperation on both continents, and in this country under the vigorous leadership of Dr. Hognestad, the translation of important CEB documents into English is well underway.

Also in its meetings and conventions ACI is moving toward international cooperation.. In 1956 the regional meeting was in Montreal, Canada, with an attendance of 486, a near record. Strictly speaking this was our first international regional meeting (there was an annual meeting in Toronto in 1934), although Canada has always seemed more than a neighbor and its engineers have been most active in ACI. Many of you will recall the late R. B. Young as a past president (1940) and later an honorary member in 1953. Rather recently Hugh C. Ross of Toronto was an ACI director. Now Canadian members are on the Building Code committee and many other ACI committees.

From our southern neighbor Mexico, Señor Barona de la O has served as a director, and the successful regional meeting, last November, in Mexico City is still pleasantly fresh in our memories. Dr. Rosenblueth

of Mexico City is a member of the ACI Building Code committee. A regional meeting to be held in Caracas, Venezuela, has recently been under discussion as an extra meeting, not as a substitute for a regional meeting in this country.

A Spanish version of the Committee 616 report *"Guide for Painting Concrete"* and the ACI Standard on portland cement paint, ACI 616-49, was approved by the Committee on Spanish Translations, consisting of ACI members in Mexico, Panama, and Peru, and published cooperatively with the Instituto del Cemento Portland Argentina. The committee is now reviewing translations of the ACI Building Code and the *Reinforced Concrete Design Handbook*. Interest has been expressed in translating the *ACI Manual of Concrete Inspection*. An Italian translation of the *Concrete Primer* has been printed and distributed, and permission has been given for a Norwegian edition.

The incoming president, Prof. Joe W. Kelly, has a keen interest in international relations and is currently chairman of a special committee studying how ACI can be of more service to members outside the United States. It is definitely true that ACI does not want to compete with or be detrimental to any engineering society in other countries. Yet ACI officers see large advantages to all from the wider exchange of information and papers between countries all over the world. As of today ACI is the most available medium for this international interchange.

ACI COMMITTEE OPERATIONS

Those who thought they had no interest in the committee sessions of yesterday and today have overlooked a rich portion of the convention. These open committee meetings are among the most inspiring sessions of the convention, giving the members a chance to hear what is developing before it is ready to be put into print. Started as an experiment not many years back, the full day and a half for committee meetings has developed into the high point of the convention for many. If you missed it this year, make a note to include these days at the convention in St. Louis next year.

The ACI philosophy of committee work has been changing over the years. ACI has many committees because technical progress either originates in these committees or the new developments are there given the critical review necessary for public acceptance. Some of these committees are semi-permanent in nature, but committee chairmen are given only a 3-year appointment and are expected to achieve results in that time. Although a 1-year reappointment is possible and is often made under special conditions, normal procedure calls for the chairmanship to move to another man at the end of 3 years. The ex-chairman is expected to continue as a committee member to maintain continuity in committee thinking.

Under ideal conditions many committees should complete their mission in 3 years. But committees work on donated time and without travel budgets. Hence they proceed somewhat slowly at times because of the inertia associated with correspondence. Nevertheless, when a chairman fails to make his requested annual committee report, this usually means little work has been accomplished. Such an omission automatically places the committee on the agenda of the Technical Activities Committee and raises the question as to whether the committee should be discharged, a new mission given to it, or simply a new chairman appointed.

ACI has no honorary committees. If a committee has completed its assignment and has no immediate task ahead, the committee is discharged and only the committee's name, mission, and last activity are recorded in the directory. Such a committee can be reactivated whenever a new task appears, as happened this past year when Committee 505, Reinforced Concrete Chimneys, was reactivated to study the special problems created by the increased use of forced drafts in chimneys.

Under this system the number of committees has considerably increased until now there are 51 committees, many with their own subcommittees. Some 580 individuals serve on these committees and many of these work on more than one committee. Your president is awed at the number of man-hours given by these busy people in developing committee reports and standards. While we are indebted to these gentlemen, we can also say that they are solving their own problems by cooperating with others. I venture to suggest, if any ACI member has a problem which he believes would profit from committee study, that a letter from him might activate a new committee and possibly even give him a new committee job.

Committee membership is an honor. Committee members have the privilege of presenting their ideas for critical review and of being among the first to learn of new developments and ideas in their field of interest. But committee membership is never honorary; it implies the duty of contributing to committee work. If a member fails to answer his mail, he may quickly become an ex-committee member.

As a result of committee work, publications and standards have kept pace with the general ACI growth. In 1959 the JOURNAL consisted of 2220 pages compared to 1490 pages as late as 1954. Today we have scheduled the presentation of three new committee reports and if you scan your program you will note that other committees are represented.

CONCRETE INDUSTRY PROBLEMS

One should not gather from the foregoing that the work of ACI is complete nor that the concrete industry has reached the place where it can be content with past honors. In many areas in the industry

conditions now exist that are wholly unsatisfactory. Without attempting to exhaust this list I shall refer at this time only to three: uniformity of concrete strength and properties, adequate inspection of construction, and adequate research.

Strength uniformity

The industry is well aware that concrete strength, under properly controlled conditions, can be maintained reasonably uniform and quite close to the desired level. The ACI Building Code committee is ready to include procedures which utilize all of the dependable strength of concrete. However, the Code committee knows and the industry knows that it is not likely to obtain uniform strength unless the engineers on the various jobs exercise more than the present usual care in supervision. As a matter of fact, variation in concrete strength on the job is one of the most costly factors now entering into concrete construction. It does no good to have an average strength of 3000 psi on the job if this means that some strengths, because of loosely controlled procedures, are as low as 1500 psi. The speaker has no easy solution for this problem except to caution designers to consider this concrete as 1500-psi concrete and not 3000-psi concrete.

In some way the industry must come up with a better system of concrete mix control than it now uses. The present occasional job where control is good must become the rule instead of the exception.

Engineering inspection

A second problem lies in the lack of proper engineering inspection and this is more of a matter for the architects and the engineers than it is for the construction man. However, it must be clearly recognized that it is a wasteful process to make designs carefully if the architects and engineers are to wipe their hands of the project at that stage and leave it to be constructed without supervision or under poor supervision. I am afraid that too many design engineers are leaving their reputations entirely too much in the hands of the steel setter. These men are not technically trained and it is not surprising that one hears repeatedly of jobs in which the bottom steel was placed on top and the top steel was placed on the bottom. Obviously this can be quite destructive to an engineer's reputation. He must follow with inspections to see that such events either do not occur or are corrected.

Inadequate research sponsorship

Finally there is the matter of lack of research in the field of concrete and reinforced concrete. It is not implied that research is totally missing, but it is implied that, in comparison to the need, very small efforts and very small funds are being directed toward research. If the construction industry feels that engineers know the proper answer to the technical

questions on which they must act, a visit to some of the technical committees, such as the committee on prestressed concrete, the committee on composite construction, or just to the ACI Building Code committee, might be most revealing. There are many questions crying for answers and, speaking generally, there is all too little money with which to provide the answers.

The chemical industry finds it a good investment to put from 1 to 4 percent of its gross sales into research, the average being about 2.8 percent in the last report I have. I doubt that the construction industry as a whole averages 0.1 percent of its funds directed to research. In my own state where the highway department expenditures run close to $400 million a year, I doubt if anything near a single million goes into physical research. It is a fact that some other state highway departments are more progressive in their reaction to the needs of research but it seems to your president that very few put real emphasis on research. I wonder if we can hope to meet the present problems and the problems of the future with something like a penny out of every $10.

Some product manufacturers do research directly related to their product but this is not the general type of research which is necessary for the concrete industry as a whole. Certain trade organizations contribute in a modest way to research projects, but if you scout over the country you will find only a handful of universities doing any subsidized research and another 12 or 15 probably work only at the graduate thesis level without much, if any, financial encouragement.

I am sure that everyone in the audience is now sitting back very comfortably saying to himself that this is not an ideal situation and "George ought to do something about it.." Since George seems to be absent, I want to raise the question as to what we as individuals are doing about research and what are we willing to do?

If I should ask the architects and consulting engineers in the audience to raise their hands if they had ever contributed as much as $100 to concrete research, it would startle me very much if as many as three hands went up. If I asked the contractors in the audience how many of them supported a regular research program, or for that matter had ever supported a research project, it would surprise me if the response was much greater. I know I would be wasting our time if I asked those of us who are on a salary how many have made money contributions to research.

One committee chairman who works diligently to get research money for other people's projects has stated that one almost has to get down on his hands and knees to get any response from government agencies and that even these methods do not work with industry. This leaves the trade associations and a few universities largely responsible for the research progress of concrete in this country.

The Portland Cement Association in its Skokie laboratories is doing some excellent work and expending some substantial funds. I am sure that a laboratory of this kind is the efficient way to get immediate returns from the funds that are expended. Nevertheless, I venture a question to the Portland Cement Association. I wonder if $100,000 or $200,000 of their budget might not in the long run be more effectively spent if it were used to encourage the beginning of research in universities where little of such work is now being done. I know the returns at first would be small, but I wonder if the ideas from these independent groups might not lead to even more original approaches and greater advances in the long run than when all the funds are used in one central laboratory. Certainly such financial encouragement would lead to much more interest in the field of reinforced concrete and many more student theses on this subject.

I do not think that the construction industry can afford to pin its hopes of the future entirely on the work done by the Portland Cement Association and a few university laboratories financed as at present. It seems that the American Concrete Institute should do something about this. However, it should be obvious to you, as it is to those of us who have considered the matter, that 10,000 members paying in dues of less than $200,000 a year are not going to provide for the running of the Institute, the publication of the JOURNAL, and a substantial research program. The American Concrete Institute can only enter the research field in a different fashion.

In the Southwest, football is taken rather seriously and in my state there are any number of alumni football clubs under various names. The men in these clubs do not hesitate to go all the way across the state to interview a good football prospect. If conference rules did not prohibit it, they would give much more than their time in trying to bring good football talent to their home schools. What we need in the American Concrete Institute is not a "football booster club" but a "concrete research booster club," which we might have to call something more dignified. If such a committee showed as much interest in research as the football booster club shows in football, it would not be long in making its influence felt. After all, the American Concrete Institute and the concrete industry have no conference rules against the use of funds for research. The American Concrete Institute has officers and directors who would see that such funds were carefully and wisely spent in the development of this great construction material.

I hope that you do not take this entirely as a fanciful idea. I believe that the concrete industry has been exceedingly fortunate to progress to its present state with the very small outlay that it has made for research. Competing industries are spending considerable sums in developing their products. If the concrete industry is to survive and

develop and avoid the designation of "the industry that used to be important in construction," I believe that it must face up to the fact that its design engineers and its construction units have never been very seriously concerned about the development of better methods through research. We must realize that research is a profitable investment, not an expense.

CONCLUSION

ACI, in your president's opinion, is not just another technical society; in two respects it is almost unique. Like other societies its meetings provide fellowship for its members, but more than in any other group your president knows, there is a special spirit of comradeship and sharing in ACI. Here the amateur in any area can ask the most eminent practitioner a question and probably get a personal interview. The spirit of the convention is equally as much to share one's information as to acquire new ideas.

The second difference one quickly notes is that the ACI staff is anxious to help get things done. Headquarters does not start by looking for some rule against what needs to be done.

Gentlemen, the presidency of the American Concrete Institute is a high honor, and I am most grateful for the confidence that you have given me. Your organization is exceptionally strong and even a poor president would have considerable trouble in interfering much with the progress of the Institute. It has been a real privilege to be able to observe the wide scope of activity in the Institute and in the industry; and in this respect the presidency is a most challenging position. I am certain that the role of the American Concrete Institute in the future is going to be an expanding role and that it will have your backing in all the moves it takes in that direction.

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Title No. 56-55

Research, Building Codes, and Engineering Practice

By CHESTER P. SIESS

The function of research and practice as sources of knowledge, and the utilization of this knowledge in the preparation of codes or specifications are considered from a philosophical and historical point of view. Examples drawn from current or past ACI Building Codes are cited to illustrate the roles played by research and by engineering practice in the drafting or revision of codes.

■ THIS PAPER WAS PREPARED ORIGINALLY for presentation at the 12th Regional Meeting of the American Concrete Institute in Mexico City. Since the engineers of Mexico City were at that time preparing the draft of a new building code, it was felt that some discussion of the interrelationships among research, building codes, and engineering practice would be of interest.

Those readers who have experience in research or who have been engaged in writing codes or recommended practices will undoubtedly find little new among the ideas or information presented in this paper. On the other hand, it is hoped that engineers in practice will find something of interest in this discussion of the interrelationships and flow of knowledge among research, building codes, and practice.

None of the remarks or conclusions in this paper should be taken as representing the official or collective viewpoint of ACI Committee 318, Standard Building Code, or any other specification or code writing body with which the writer has been associated. It is neither necessary nor desirable that all members of such bodies approach the task of writing a code from the same point of view or with the same philosophy. The views expressed herein are solely those of the writer. It is hoped that other viewpoints will be presented in the discussions.

THE FLOW OF KNOWLEDGE

The practice of engineering as a profession is, by its very nature, based on knowledge. There are basically only two sources of this knowledge: research and experience. Let us now consider how this

ACI member **Chester P. Siess** is professor of civil engineering at the University of Illinois where he has engaged in research on reinforced and prestressed concrete and highway bridges since 1941. He is a member of ACI Committee 318 and was also one of the authors of "Tentative Recommendations for Prestressed Concrete" prepared by ACI-ASCE Committee 323. For a number of years he has cooperated informally with the AASHO Committee on Bridges and Structures in connection with the preparation of specifications for highway bridges.

knowledge is or can be transmitted from its source to the potential or practicing engineer.

There are two separate phases or periods in the engineer's life which must be considered. The first is the period of formal education, at the end of which the engineering degree is obtained. In this stage, the sources of knowledge to the engineering student are chiefly indirect in that he receives his knowledge almost exclusively from his professors and from his textbooks; relatively little of it is obtained from direct participation in research or from experience. The teacher may have obtained his knowledge directly or indirectly. That is, he may be a research man or a practicing engineer himself, and may transmit to the student knowledge he has obtained at first hand. Or, alternatively, he may have devoted considerable time to the study of the results of research and collected and digested this information for transmission to his students. In many cases, the result of research and the experience from practice have been summarized in textbooks from which the student may gain his knowledge under the guidance of his professors.

This entire process through the period of formal education, however, has one limitation. The knowledge gained by the student can be no more than that which exists at the time he graduates. Actually, in most cases, the schools and professors cannot keep completely up to date, and the knowledge received by the student is probably at least 2 or 3 years old. So far as the basic mathematical and engineering sciences are concerned, knowledge is probably timeless, but to the extent that engineering is a continually expanding profession, new knowledge is being added every day. This means simply that the engineer must keep on learning, even after he has received his degree, and the procedures for acquiring knowledge in this second stage of his career must now be considered.

The sources of knowledge are still the same: research and experience. Since the engineer is now practicing, some of his additional knowledge will come from his own experience, or perhaps from his own research if he is in a position to do any. This is not enough, however, since he must also be able to benefit from the research and experience of others in the profession.

If we assume that the results of research and fruits of the experience of others are made public in technical papers, then it is at least theo-

retically possible for the individual engineer to increase his knowledge by going directly to these sources of information. This of course is a most desirable procedure, and there are undoubtedly many engineers who have the time, skill, and patience to make detailed studies of research reports and technical papers, to dig out the significant bits and pieces of knowledge, and to apply the results directly in their engineering practice.

However, for all but a few really great engineers, it would seem that the process of individual study of the results of research or of technical papers based on experience might well be replaced by an alternate procedure in which a small group of engineers is formed into a committee to study collectively the results of research and current practices. This group then, through its collective judgment, can prepare a summary of existing knowledge which is incorporated into a set of recommendations and rules for design and construction, which we know as a specification or a code. We see then that building codes are, or can be, a powerful and effective means of collecting and disseminating knowledge. This knowledge comes from research and from experience gained in practice. It is used in practice and, of course, it is also used in the engineering schools.

This is one reason why a code is frequently interposed between the results of research and their application in practice. Another reason is that codes are nearly always restrictive in one sense or another, and it is frequently found that the results of research can not be applied directly by the practicing engineer because they lead to a design that is not permitted by the code. In such cases, the results of research may have to influence the code before they can influence practice.

The complex flow of knowledge among research, codes, and practice may be illustrated rather crudely by the flow diagram shown in Fig. 1. This is a complex flow diagram but a realistic one.

Practice of course came first, and for a time it was complete within itself. That is, knowledge gained from experience in practice fed back on itself as indicated by the little loop. This is still true today, but only to a limited extent. The individual engineer can of course utilize directly in his own practice the knowledge gained from his own experience, provided that in doing so he does not violate some provision of the existing codes. On the other hand, the individual engineer can utilize directly the knowledge gained from the experience of others only if their experiences have been published in technical papers or textbooks and if he has the ability and patience to study those sources and absorb and apply the knowledge contained in them. In most cases, knowledge from the experience of others flows first to the code-writing body and then back into practice, as shown on the diagram.

Similarly, the knowledge gained from research ideally should reach practice directly. But under our present system, it is much more likely to flow primarily to the code where it is blended with the knowledge from experience before being made available and useful in practice.

Although this interposition of a code between the sources of knowledge and the people who must use that knowledge in practice is undoubtedly restrictive, it is not necessarily a universal evil. Although it is most desirable that knowledge gained from any source be utilized as fully as possible and for the benefit of all, it is equally important that this knowledge be used correctly and that its misuse not be permitted. For example, all research is not equally good; all research results are not equally valid; and much research cannot be applied directly to practice except by an engineer having considerable experience and possessing a sense of caution based on that experience.

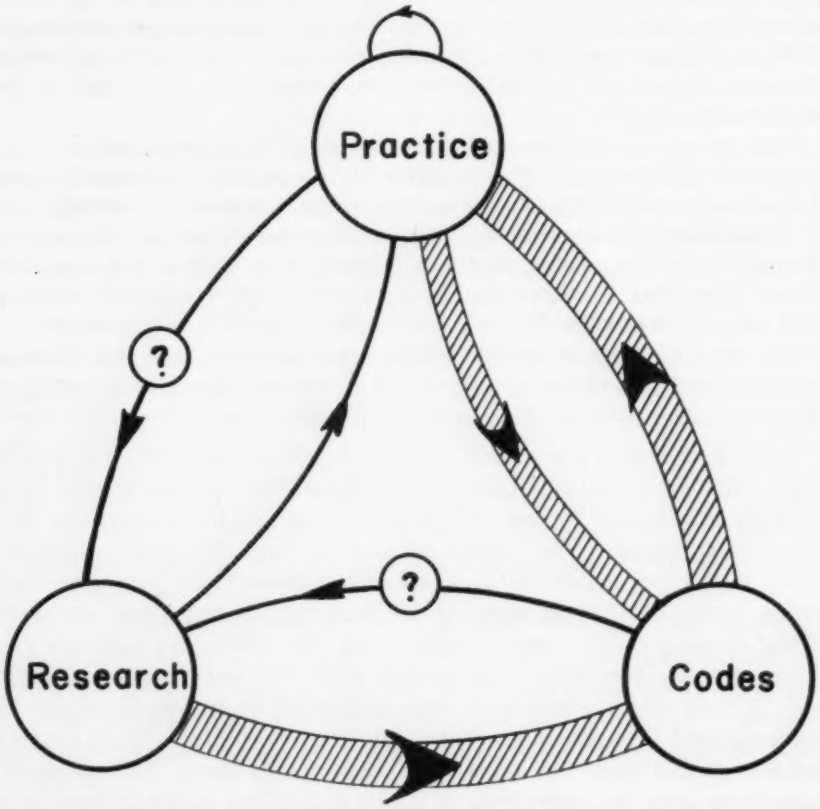

Fig. 1—Complex interrelationship of research, codes, and engineering practice

Similarly, the knowledge gained from experience is frequently limited in scope, and there is always the possibility that it may be extrapolated to applications for which it is not valid. In many instances, the role of research is to determine the range of applicability of knowledge gained from experience, or to extend that range, or to define its limits so that it can be used safely by all.

Thus, we see that the meeting of research and practice in the code is potentially capable of deriving the best from each and presenting to the profession a synthesis of contemporary knowledge and current practices.

Returning to the flow diagram, we see that the paths of knowledge which flow from research and from practice, through the code, and back into practice are the main channels. There are also supplementary channels, particularly from research directly into practice, and the little feedback loop from practice back on itself, without going through either research or codes. There is also an important feedback from practice to research and from the code to research; this is primarily in the form of questions which arise in practice or in the writing of codes and which are referred back to research for answers.

This feedback of questions is a most important aspect of the flow of knowledge, since our research can never be better than the questions we set out to answer. It is essential that we ask the right questions and ask them in the right way. As Hardy Cross has said "any fool can ask a question that the wisest cannot answer" but "more important . . . only the very wise can ask questions in such a way that any fool can answer them. If the questions are good questions the answer can probably be found and if they are poor questions no one can answer them."[1]

It is doubtful, however, whether this particular aspect of the flow of knowledge and interrelationship between research, codes, and practice is of much interest to any except research men and it will not be discussed further in this paper. Instead, let us now take a closer look at how the knowledge gained from research and from experience is utilized in the preparation of building codes.

THE NATURE OF BUILDING CODES

First of all it is worthwhile pointing out that most building codes, at least those that the writer is familiar with, are entirely empirical in nature. That is, they are based on experiment and experience. The experiments have usually been carried out in the laboratory and are thus a product of research. However, experiments also may be carried out in the field, either deliberately or by accident. In either case, it is probably not too much of a generalization to say that the knowledge gained from experiment nearly always relates to the strength of members or structures. On the other hand, our experience usually comes

from practice. But we must not overlook the fact that the research man who has spent most of his life working in the laboratory has also developed experience. Again, one can generalize by saying that most of our experience, whether obtained in practice or in the laboratory, relates to the behavior or performance of members or structures; that is, to their serviceability, rather than to their strength.

The statement that our codes are primarily empirical in nature does not necessarily mean that all of our knowledge comes from experiment and experience, and none comes from theory. We use many theories in the design of reinforced concrete structures, but such theories are usually assumed to be a part of the body of knowledge of the profession — something that has been learned in school — and they are included in the code only by reference. For example, Section 601 of the current ACI Code, under the heading "Design Methods," refers to the "accepted straight line theory of flexure." In Section 701(a), reference is made to design for maximum moments and shears "as determined by the theory of elastic frames." And in Section 1002(a), relating to flat slabs, there is a reference to "recognized elastic analysis."

It can be seen then that the ACI Code at least recognizes the existence of certain theories and refers to them or actually requires their use. However, this does not necessarily mean that the design procedures of the code are rational simply because they are based on what we would consider to be rational theories. The reason for this is simple. All theories are based on certain assumptions, and their use is justified only to the extent that these assumptions are correct. The theories used in connection with reinforced concrete are nearly always based on assumptions regarding the properties of the materials or the properties of the structure that are not even approximately correct for reinforced concrete. This is certainly true of the "accepted straight line theory" and the "theory of elastic frames" or the "recognized elastic analysis" as applied to reinforced concrete members, frames, or slabs. Nevertheless, the use of these theories in connection with the design of reinforced concrete members or structures has been justified over a period of years by experience or by experiment. In other words, we have learned how to apply an elastic theory to an inelastic material or structure and still obtain a safe and serviceable structure. To this extent, then, all reinforced concrete design, and practically all aspects of our building codes, even where they involve theories, are basically empirical in nature; that is, they are based on experiment and experience; on the knowledge gained from research and from practice.

We may now consider a few specific examples taken from the ACI Building Code to illustrate the complicated interrelationships between research and practice in the writing or revising of a code.

FLOOR SLABS

One of the best examples is afforded by the Code provisions for the design of reinforced concrete floor slabs. The current ACI Code provides design criteria for two basically different types of floors.

The so-called two-way slab is covered by Section 709 in Chapter 7 on flexural computations. This section consists of four paragraphs, followed by 5 pages of detailed criteria divided into two parts designated as Method 1 and Method 2.

The other type of floor system covered by the Code is the flat slab, and design criteria are presented in Chapter 10, devoted entirely to this type of construction, and taking up about 14 pages.

The Code provisions for these two types of slabs are vastly different in their origin, subsequent development, and present form, as well as in the type of design they yield.

The flat slab as a type of construction was not a child of research. It was developed in practice over 50 years ago when reinforced concrete was in its infancy. Actually, it was invented, and the idea or system of construction was patented. At first, flat slabs were not designed; they were simply built. Although this procedure was acceptable in the days of the great cathedrals, it was not completely acceptable in the modern world with its emphasis on theory and a rational approach. Both the owners of the buildings and officials of the city governments were skeptical of structures for which no design calculations could be shown. Consequently, most flat slab floors were sold on a "money back" guarantee that they would meet the requirements of an actual load test without exceeding a certain specified or guaranteed maximum deflection.

The flat slab was an immediate popular success, either because of or in spite of the fact that its design was not covered by any existing building code. The principal reason for its success, of course, was its great economy, especially for floors designed for heavy loads. By 1913, only 7 years after the first flat slab floor had been constructed, it was estimated that more than 1000 buildings had been constructed using this system. This constituted 80 percent of all buildings designed for a live load greater than 100 psf.

During this period of intense construction activity, the profession began to ask questions, and many theories were developed to explain or justify the designs used. At the same time, a number of thorough load tests were made on actual buildings. Although these tests were much fewer in number than the usual acceptance tests, they were much more comprehensive in scope, involving hundreds of strain measurements to determine the magnitude and distribution of strains, stresses, and moments. This then was a period of research, both theoretical and experimental, which continued to about 1920 or 1921.

The first really useful theoretical analysis was that published by Nichols in 1914,[2] which provided for the first time a means of computing the total statical moment which must be resisted by a flat slab. This theory, however, came too late to influence significantly the first design provisions for flat slabs which appeared in the building codes. The first code was probably that of the city of Chicago in 1914. This was followed by code provisions for flat slabs in the report of the First Joint Committee in 1916 and subsequently in the various ACI Codes.

All of these codes had one thing in common: the total moment for which provision must be made was less than the total static moment derived by Nichols. The 1916 Joint Committee Report[3] provided for a moment equal to 85 percent of the static moment, whereas the ACI Codes from then down to the present time provide for a total moment equal to only 72 percent of the static moment. These code provisions were obviously based on experience gained in practice and not on research, although much of the early field research involving extensive strain measurements had been interpreted by some to justify designs based on less than statics. This interpretation, however, depended solely on the assignment of a certain fraction of the load-carrying capacity to tension in the concrete.

It seems possible that if Nichols' analysis had been available earlier, and if we had understood better the great reduction in steel stress that can be attributed to tension in the concrete in slabs having very small amounts of reinforcement, that the early codes might have been somewhat more conservative than they were. That is, they may have called for a design based on 100 percent of statics. But this is a case in which practice came first, and the designs developed in practice became well entrenched in the code before the confirming or nonconfirming research was carried out and reported and interpreted.

Actually, relatively little physical research on flat slab floors has been carried out since the design procedures were more or less frozen in the codes around 1920. And this research has played a relatively small role in the code provisions for flat slabs. Even where research has demonstrated deficiencies in design procedures, little or no change in the code has taken place, primarily because there has been no demand for such a change from practice. It has been the experience of the profession that flat slab floors, designed according to the code, behave satisfactorily. That is, they do not fail, and they do not deflect excessively. Current research, therefore, is aimed not so much at revisions to the code but at explaining the reasons why structures designed on such a basis have in fact performed so well.

Now let us consider the two-way slab; that is, the slab supported on four sides and reinforced to carry moment in two directions. We have always had this type of construction. It was not invented, but was

simply a rational development. We see then it too developed in practice. But, in contrast with the flat slab, the code provisions for two-way slabs developed primarily out of research. And, more important, these provisions were based primarily on analytical research rather than experimental research.

As compared to flat slabs, very few tests have been made on two-way slabs, either in the laboratory or on actual structures in the field. On the other hand, extensive studies have been made of the moments in such slabs, particularly in the structure involving many panels with slabs continuous over the supporting beams. In the United States, and in the ACI Code, this research has followed two paths, each leading to one of the methods of analysis and design now specified in the Code.

Method 1 of the Code was based directly on analytical studies by DiStasio and Van Buren which were reported in a paper in the ACI JOURNAL in 1936.[4] Their studies were almost entirely analytical in nature and dealt only indirectly with the results of the few tests available.

Method 2 entered the ACI Code by way of the 1940 report of the Third Joint Committee.[5] The provisions in the Joint Committee Report were in turn based on a most interesting and extensive analytical study published by Westergaard in 1926.[6] Westergaard's proposed method of analysis and design was based predominantly on theoretical analyses of moments in continuous plates, but also took into consideration certain aspects of behavior which had been observed in tests. Nevertheless, the step from Westergaard's 1926 paper to the 1940 Joint Committee Report was not a direct one. His proposals were extensively modified and simplified by the Joint Committee and it might be considered here that these modifications of the results of research were based on experience in practice. Unfortunately, there is no record of the basis for these modifications, and the extent to which his recommendations were influenced by practice can only be guessed at.

The result, however, is two sets of design criteria for two-way slabs, Methods 1 and 2 in the ACI Code, which are both much more conservative than those for flat slabs. In effect, designs must be made for maximum positive moments obtained under one condition of loading and for maximum negative moments obtained under another condition of loading. As is known, this can lead to a total moment-resisting capacity significantly greater than that required by statics.

We thus have in the ACI Code a most interesting situation. For flat slabs, we have a design procedure based almost entirely on experience in practice, and requiring a design for 20 to 28 percent less moment than that indicated by statical calculations. On the other hand, we have for two-way slabs a design procedure based almost entirely on theoretical research, and requiring a design for moments totaling 20 to 30 percent more than those obtained by statical calculations. This apparently

inconsistent approach to the design of reinforced concrete slabs has existed for well over 20 years and it is likely that it will exist for at least a few more years before the Code is changed in connection with one or the other of these two methods of design. So far as flat slabs are concerned, it seems extremely unlikely that any research will be able to contradict the experiences of practice and cause a significant increase in the moments for which flat slabs are designed. On the other hand, experience in practice has provided no basis for a reduction in the design moments for two-way slabs. Experience in practice usually can show only deficiencies in design, it is most difficult to determine from the observed satisfactory performance of a structure how much weaker it could be made without getting into trouble. It might almost be said that changes in the Code to remove this inconsistency are hampered both by experience and by tradition, and not least by lack of adequate research. It is worth while mentioning, however, that this lack of research is being remedied and that there is now underway at the University of Illinois a program of laboratory tests which may throw some light on the comparative strength and behavior of flat slabs and two-way slabs. Whether the results of this research will lead to changes in the Code, and what direction these changes might take, can not be predicted. As has been indicated so far in this paper, and as will be brought out with further examples, the impact of research on the Code is not something that can be predicted without taking into account the extremely strong influences from practice itself.

COLUMN FOOTINGS

Column footings provide another example which is similar in some respects to the case of the flat slab. The present ACI Code provisions for isolated column footings provide for a design based on only 85 percent of the static moment. This provision of the Code also may be traced back almost 50 years, but it had its origin in research rather than in practice as was the case for the flat slab.

The extensive program of tests on footings reported by Talbot in 1913[7] led to the so-called trapezoidal rule for the calculation of moments in footings. This rule was complicated in its origin as well as in its application, but in effect it resulted in the calculation of a moment less than the total statical moment of the forces on one side of the critical section. Unlike the flat slab, which is a highly indeterminate structure, the simple column footing is easily analyzed and it was obvious almost from the beginning that the design was based on less than the static moment. If this were not enough, the test results reported by Richart in 1949[8] proved conclusively that all of the static moment was resisted by tension in the reinforcement.

In the 1941 ACI Code, the trapezoidal rule was dropped from the Code and replaced by a much simpler method of computing the moment. But as a concession to the previous long established practice, the designer was permitted to proportion the reinforcement for only 85 percent of the static moment. At the time of the code change in 1941, and at several times since, serious attempts have been made to change this provision of the Code to provide for 100 percent of the static moment. It has been argued that this change will add negligibly to the cost of a footing and almost infinitesimally to the cost of a structure. It has also been argued that the present Code provisions are extremely difficult to explain to engineering students and the plight of the university professors would be helped by a more rational and statically correct procedure. It may be noted here that this difficulty of teaching design for less than statics does not seem to arise in connection with the flat slab floor, which is a much more complicated structure and one in which the abnormality is not so evident to the student. In any case, every attempt to change this provision of the Code has been strongly resisted by engineers in practice. Experience in practice has apparently been good; that is, again there have been no failures and no examples of poor performance. This argument, however, may be somewhat weaker in the case of footings than in the case of flat slabs since failures in footings are almost universally attributed to the soil rather than to the structural concrete. But be that as it may, this is another example of a Code provision, originating in research, and presumably confirmed by practice, becoming entrenched from a practical point of view. It is doubtful whether research alone will ever produce a change in this provision of the ACI Code.

SHEAR IN BEAMS

Two of the examples just cited, the case of the flat slab and the case of the isolated column footing, are excellent illustrations of circumstances in which good performance, or what is thought to be good performance, has provided the reason for keeping in the Code provisions which are not fully supported by subsequent research. Knowledge gained from experience, in the form of knowledge relating to satisfactory performance, has outweighed knowledge gained from research. But this influence of knowledge gained from experience works in both directions, and another case can be cited where the knowledge of poor performance — in this case, actual failure — was the basis for revising certain provisions of the Code which presumably were based originally on extensive research. The case in question is the addition of paragraph 801(e) to the 1956 ACI Code, a change which was made chiefly if not solely because of failures observed in the field.

The ACI Code requirements for shear have not been changed significantly in the past 30 years. Since a tremendous amount of research work had been carried out prior to 1928, it is reasonable to assume that these provisions of the Code are based primarily on that research. Although it is reasonable to make this assumption, in many cases it is quite difficult to show a direct relationship between the requirements of the Code and the results of that research. Since 1945 there has been a resurgence of interest in research on strength in shear. Certain deficiencies in the previous research, primarily in the nature and the range of the variables, have been pointed out and much new research has been undertaken. At the present time, important revisions to the Code are being considered as a result of this new research. But the revision in Section 801 was not based on this new research but solely on an unfortunate experience in practice.

In August of 1955, portions of three rigid frames supporting a warehouse roof in Shelby, Ohio, failed in shear. The failure was rather sudden, as is typical of shear failure, and about 3000 sq ft of roof collapsed completely. The failures occurred in the continuous girders near the points of inflection and at locations where the computed shearing stresses were quite small. Subsequent investigation revealed that the design of these girders satisfied all provisions of the 1951 ACI Code. Any structural failure is a cause for concern in the profession, but this failure was of even greater importance since the structure which failed was one of perhaps 20 similar warehouses of essentially the same design which had recently been built or were then under construction. Literally hundreds of almost identical continuous frames were involved, and the Corps of Engineers of the U. S. Army, under whose direction these structures had been designed and were being constructed, lost no time in initiating a complete investigation into the causes of the failure. This investigation was carried out by various consultants and was aided by an intensive series of tests on 13 one-third scale models, carried out with great expediency by the Research and Development Laboratories of the Portland Cement Association. Although these investigations did not provide a complete explanation of the causes of this failure, they did show how similar failures in similar circumstances could be prevented in the future. And by February, 1956, less than 6 months after the failure occurred, Section 801(e) had been inserted in the ACI Code. This section is intended to prevent such failure by requiring the use of web reinforcement in the neighborhood of the point of inflection of continuous girders, even if the computed shearing stress is less than that previously allowed in a beam without web reinforcement.

This is clearly a special case, since all failures in practice should not necessarily lead to such immediate changes in the Code. The fact that

several hundred similar continuous girders were involved certainly had a bearing on the urgency of the problem. This was especially true since careful inspection of these other girders showed definite signs of distress in many many instances. It is doubtful whether such an extensive investigation would have been made if only one girder, or even only one structure, had been involved.

There are two things to note in connection with this incident. First, the change in the Code was so written as to apply solely to the type of construction which failed, and the requirements for web reinforcement were essentially the absolute minimum necessary to prevent such failures in the future, as determined from the program of laboratory tests. And second, this is a case where the Code was made more restrictive.

In general, changes in a code tend to make it less restrictive, and it is usually extremely difficult to make the code more restrictive on the basis of research alone. This has been demonstrated by the example cited in connection with flat slabs and footings. It is always easy to relax the code provisions on the basis of research and this is perhaps as it should be. The code should initially be relatively strict, in view of our ignorance, and the provisions should be relaxed as our knowledge increases. As a result, to make the code more restrictive we require either a tremendous volume of almost incontrovertible research, or one or two good failures in practice.

Ideally, we should never have failures, at least not those attributable to deficiencies in the code. Codes are not infallible, even when based on research, since research itself is not infallible. Research is not infallible primarily because it is never complete; that is, although laboratory tests permit more careful control of the variables than is possible in the field, the number of variables included in tests is usually much smaller than those actually existing in the structure. We test simplified, even over-simplified, representations of real structures. For example, although many beams had been tested, including beams with simulated or actual continuity and including rigid frames, nothing exactly comparable to the frames of the warehouse that failed had ever been tested in the laboratory. Consequently, code provisions based on this research were deficient in at least one respect.

DEFORMED BARS, BOND, AND SHEAR

It is not always best to apply the results of research directly to building codes or to practice. The research itself may be good, but the results may be used improperly. An example of this might be the changes made in the 1951 ACI Code as a result of the development of new types of deformed reinforcing bars. These changes consisted chiefly of changes in the allowable bond stresses, and these were probably quite legitimate since they were based on extensive tests of bond. However, at the same

time, and somewhat indirectly, a change was made in the allowable shear stresses in beams without web reinforcement.

In the 1947 ACI Code, two different values of the allowable shear stress were specified for beams without web reinforcement. For beams with no web reeinforcement and without special anchorage of the longitudinal reinforcement, the allowable shear stress computed in the conventional manner was $0.02\ f_c'$. For beams without web reinforcement but with special anchorage of the longitudinal reinforcement, the allowable shear stress was $0.03\ f_c'$, or a value 50 percent higher than that for beams without special anchorage. The requirement for special anchorage of the longitudinal reinforcement was that every bar should be terminated in a standard hook in a region of compression or should be bent across the web and made continuous with the reinforcement resisting moment of opposite sign. This type of anchorage was required in order to justify the 50 percent increase in allowable shear stress.

Similarly, a 50 percent increase in the allowable bond stress was permitted if the longitudinal reinforced bars were hooked.

In the 1951 Code, the requirement of special anchorage was dropped and it was required instead that all plain bars must be hooked and all deformed bars must be of the new improved type meeting the requirements of ASTM A 305.[9] So far as bond stresses were concerned, there could be no argument with this line of reasoning. However, at the same time, the lower allowable shearing stress of $0.02\ f_c'$ was deleted from the Code and the higher stress of $0.03\ f_c'$ was left in. The implication here was that the use of new-type deformed bars without hooks, or the use of plain bars with hooks, was an effective substitute for the previous special anchorage requirements. If we recall that special anchorage required that all bars be anchored in a region of compression or bent across the web and made continuous with the reinforcement resisting moment of opposite sign, this assumption is certainly questionable. Under the 1951 and subsequent Codes, deformed bars without hooks, or plain bars with hooks, can be terminated in a region of tension. There are no directly applicable data from either research or practice to indicate that this increase in allowable shearing stress has led directly to any failures in shear, but it is entirely possible that the warehouse girders mentioned previously would not have failed if they had been designed according to the provisions of the 1947 Code.

The point to be noted here is that this is an instance in which a change in the Code was based on research, but on research which may not have been interpreted or applied correctly. That is, the application of the results of research on bond, in this case, had a significant effect on the allowable stresses permitted in shear.

These examples may have given the impression that the effects of practice on the Code far outweigh the effects of research. This is not

the case; the effects of research are frequently more gradual and less obvious than the effects of practice, but in many instances they have been direct and immediate. For example, the working stress design procedures for columns subjected to axial load and bending, Section 1109 of the Code, were introduced into the Code as a direct consequence of the research published by Richart and Olson in 1938.[10] And present working stress design procedures for axially loaded columns resulted directly from the extensive and comprehensive research of the ACI column investigation in the early 1930's.

THE LAG BETWEEN RESEARCH AND PRACTICE

Actually, it must be realized that practically all of the basic information upon which practice and codes are based is originally derived from research. But the path by which knowledge gained from research eventually reaches practice or codes is frequently a long and devious one. As an example of the long time which may elapse between the attainment of knowledge and its application in the code or in practice, consider the case of the ultimate strength of an axially loaded tied column.

The law governing the strength of such columns was first stated by Considère in about 1902 or 1903 as follows: "Hence, it may be stated that in concrete members reinforced by longitudinal rods connected by cross pieces or ties too weak or too far apart to bind the concrete sufficiently together cross-wise, the total resistance to crushing varies little from the sum of the resistances offered by the crushing strength of the concrete and the longitudinal bars when stressed up to their elastic limits."[11] This is the addition law which has provided the basis for our working stress design procedures for columns since the 1936 ACI Code, and which appears explicitly in the appendix on ultimate strength design of the 1956 ACI Code. However, although this law was presumably known to research men since 1903, it was not incorporated into the Code until after it had been verified by the extensive test program of the ACI column investigation in the early 1930's.

Another such example can be taken from the history of research on strength in shear or diagonal tension. In 1909, Talbot[12] concluded on the basis of his tests that the shear strength of beams without web reinforcement depended on three factors: the quality and strength of the concrete; the percentage of longitudinal reinforcement; and the ratio of span to depth, or the ratio of moment to shear. Within a few years, however, the effects of all these variables except the strength of the concrete were forgotten or neglected, even by Professor Talbot, and were not rediscovered until the current series of investigations of shear strength were begun around 1945. The present ACI Code still relates strength in shear to only one variable, the strength of the concrete.

However, as many readers of this paper no doubt know, the results of recent research, as interpreted and studied by ACI-ASCE Committee 326, Shear and Diagonal Tension, have led to proposals for allowable shear stress formulas which involve not only strength of the concrete but also the percentage of longitudinal reinforcement and the ratio of moment to shear. Such a formula may well appear in the next edition of the ACI Code.

These examples have been chosen to demonstrate what has frequently been a great lag between research and practice, or even between research and the building code. It is not enough that we have the knowledge, we must be ready and willing to utilize it. The writer can think of almost no instance in which a breakthrough in knowledge was followed directly by a breakthrough in practice. For one reason, there have been relatively few breakthroughs in knowledge. Knowledge gained from research accumulates slowly, test by test, until one day we realize that we know a great deal more about the subject than we thought we did. When that time comes, it is frequently possible to use that knowledge to change the code, and thus to change practice. The real breakthrough comes when knowledge, new or old, is introduced into the code.

To the research man, this frequent long lag between research and practice or between research and codes is often annoying or even alarming.. At the very least, it is discouraging to realize that in many cases the knowledge gained from his work will not be applied until after he has retired. On the other hand, some lag between research and practice is probably desirable, although 30 years is perhaps too long. The question is: just how long should this lag be? Since we never have all of the answers we need from research, we can not wait for a complete solution to our problems before proceeding with the design and construction of buildings and bridges in reinforced concrete. But it is not really necessary to wait until we have all the answers. Incomplete research can be applied if it is tempered with judgement and with careful consideration of the differences between what we test, or what we analyze, and what we build. This, of course, is probably best accomplished by routing the results of research through a code to practice.

Because codes have such a great effect on practice — sometimes restrictive and sometimes stimulating — we must be extremely careful when making changes in them. Because our codes are basically empirical in nature, and are based to a considerable extent on experience in practice, we must be careful about how we appply the results of research to produce changes in the code. If a code were based only on research, it could be changed by research alone. But the codes we have today are based on a combination of research and practice, and revisions should be made only when knowledge from research and

knowledge from practice can both be brought to bear. This means, for one thing, that we should make no big or sudden changes solely on the basis of research. We should proceed in small steps and let experience in practice reveal the gaps in our knowledge, which can then be reduced or eliminated by further research.

In the proper hands, our codes lead to good structures. This has resulted from a slow, evolutionary process, utilizing knowledge from both research and practice, and has frequently involved a process of cut and try, but being careful never to cut so close that a failure occurs. As our approach to structural engineering becomes more scientific, and as design in reinforced concrete becomes more rational, it may be possible for us to go more directly from research to codes, especially if we have more basic research to supplement a more scientific or rational approach. However, at the present time, with research limited in scope, and with a relatively large gap between what we test and what we build, the step between research and practice can be bridged only with the aid of judgment gained from experience. We must continually keep in mind that the difference between a real structure of reinforced concrete and the structure that we analyze is tremendous. And similarly, the difference between a member in that structure and the test specimen in the laboratory is equally great.

RESEARCH, PRACTICE, AND CODES

If the research man sometimes has too much faith in his test results and in their application to practice, this is understandable and even forgiveable. However, the engineer with experience in practice should not fall into the same error; he should continually question the relation between what has been tested and what is to be built. The writer believes most strongly that good codes and good practice can result only from combining the knowledge from two sources: research and practice. If the practicing engineers, at least those on the code committees, can learn from research without falling under its spell, and if the research man can learn to apply his knowledge and his experience, rather than just the results of his tests, then these two groups can work together, formally or informally, to write a code which contains the best available knowledge from all sources and which will contribute to safe and progressive practice.

The foregoing remarks have been intended to give some idea of how codes are written, or how they should be written, and how they utilize knowledge from both research and practice. To illustrate certain points, examples have been taken from the ACI Code, but this should not be interpreted as any criticism of that Code. It is an excellent document and has served our needs well. Many engineers look upon codes as a necessary, or even an unnecessary evil, as something that prevents them

from doing what they want to do and what they know is right and know is best. These charges against the codes may sometimes be true but, if so, it is usually not the fault of the code or of the men who prepared it. What must be realized is this: A code is not a set of rules prepared by a few for the regulation of other engineers, but a synthesis of contemporary knowledge, practices, and techniques. Consequently, a code can be no better than our collective knowledge, whether gained from theory, research, or experience in practice.

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Title No. 56-56

Differential Shrinkage in Composite Beams

By HALVARD W. BIRKELAND

Due to normal aging and curing processes, shrinkage occurs in a slab when it is cast onto precast prestressed beams. This shrinkage induces stress into the composite beam and slab construction causing the beams to deflect. An analytical method is presented for predicting the sag. Equations are given for stresses on slab and beam section, shear and moment at the interface, and slope and deflection. This is followed by comparison of computed values with actual values obtained from full size test beams. Values are in close agreement.

■ A SLAB CAST IN PLACE ON A PRECAST prestressed beam is a technically and economically attractive structure. However, the slab will shrink and cause the composite beam to sag. Also, the shrinkage will be accompanied by stresses in the slab and the beam, and in the contact plane (interface) between the two. The engineer should predict the sag so that the slab may be screeded to a profile that will result in a reasonably level end product. He should also specify, on a rational basis, the reinforcement required to cross the interface in order that full and dependable interaction may be obtained.

An analytical method will be developed herein, and it will be tested against observed deflection of full size prestressed beams with cast-in-place slabs. A similar problem exists in the warping of stone-faced concrete slabs, and the method will also be tested against prefabricated wall panels with marble facing. It is believed that the analytical approach presented is rigorous within commonly accepted assumptions, but several uncertainties and simplifications will be pointed out.

THE PROBLEM

A precast prestressed beam is usually substantially cured and aged at the time it receives the cast-in-place slab. The curing and aging of the slab, after hardening, will cause stresses in the interface and deflection of the combined assembly. The problem is to determine the deflection of the composite beam and the stresses in the interface,

Institute member **Halvard W. Birkeland** is a partner in the engineering firm of Anderson-Birkeland-Anderson in Tacoma, Wash. The present method of analysis for shrinkage in composite construction was developed over the past 2 years from earlier and less satisfactory methods.

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consistent with interaction of the parts and equal deflections of the beam and the slab throughout the span length.

After the time of integration, the beam will continue to shrink some, there will be some additional stress creep, and Young's modulus will continue to increase. The slab is at the start fully plastic, and in the period of setting, curing, and aging it undergoes first a rise and next a fall in temperature with accompanying changes in length, and it will shrink due to drying out, all this while Young's modulus is increasing from zero toward its final value. A rigorous analysis that includes the time function would be difficult, and it is desirable to use constant values for Young's moduli and a somewhat arbitrary value for the differential shrinkage. Both of these values occur in the analytical treatment, and results of tests on composite sections should be used to establish their effective magnitudes. It will be assumed that it is possible and permissible to reduce such variable factors to a constant Young's modulus for each of the two concretes, and to a definite value of differential shrinkage. Also, only simple spans will be considered, and Young's moduli, the moments of inertia of the cross section, and the differential shrinkage will be assumed as constant through the length of the span. A vertical plane cross section will be assumed to remain plane after bending..

A solution to parts of this problem has been given by Evans and Parker,[1] who approached it by way of fiber elongations. Another approach would be to obtain the effects of shrinkage by means of self-cancelling externally applied forces. The two methods will of course give the same results. The latter method was employed by Ferguson[2] when he dealt with warping of reinforced concrete of rectangular section. His method can be generalized, and that is done herein. The approach can be described as follows (refer to Fig. 1):

(a) The beam and slab will be imagined as cast separately. The slab will shrink more than the beam, in the amount of the differential shrinkage.

(b) A pair of tensile forces is applied to the ends of the slab at its cross-sectional centroid. The forces are of such magnitude that the elongation of the slab equals the differential shrinkage.

(c) The slab and beam are brought together and integrated.

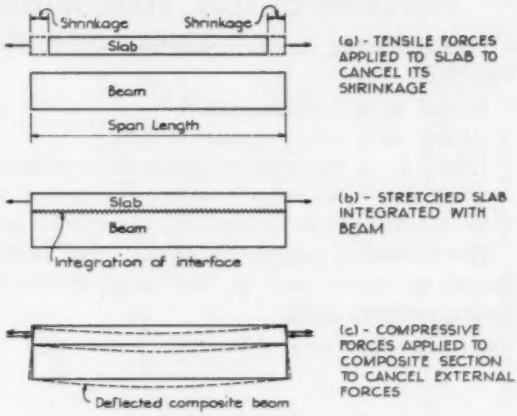


Fig. 1—The several stages in the basic concept to equal the effect of shrinkage

(d) A pair of compressive forces is applied to the ends of the composite beam at the centroid of the slab. These forces are equal and opposite to the earlier applied tensile forces, and the external forces are thereby cancelled.

(e) The resulting stresses on the cross section are determined.

(f) The shearing forces and moments in the interface and the necessary reinforcement across the interface are determined.

(g) The deflection due to differential shrinkage is determined.

Notation

Symbols are used as follows:

t = thickness of slab
d = depth of beam
e = distance between centroids
y = distance from centroid to fiber under consideration
A = area of section
I = moment of inertia of section
E = Young's modulus
ϵ = unit differential shrinkage
P = external normal force
F = internal normal force
M = external or internal moment
V = internal shear force
f = normal unit fiber stress
ϕ = angle of internal friction in interface
FS = factor of safety
L = span length
Δ = deflection

Subscripts are used as follows:

f refers to slab or flange
b refers to beam, or bottom fiber
c refers to composite section, with transformed slab
t refers to top fiber
i refers to interface, or interface fiber
s refers to reinforcing steel
y refers to yield point of steel

Double subscripts for e refer to centroids of sections

Double subscripts for y and f: the first refers to slab, beam, or composite section; the second to top, bottom, or interface fiber

Superscript $'$ refers to transformed section, but is not used for composite section which is always transformed

PROPERTIES OF SLAB, BEAM, AND COMPOSITE SECTIONS

The following basic properties are assumed to be known (refer to Fig. 2):

E_f and E_b = Young's moduli
A_f and A_b = areas of cross section
I_f and I_b = moments of inertia of cross section
t and d = thickness of slab and depth of beam
y_{ft}, y_{fb}, y_{bt}, and y_{bb} = locations of centroids with respect to extreme fibers

The following properties of the cross section are either readily determined, or were used in the usual stress computations for the beam under vertical loads:

The transformed area of the slab cross-section

$$A_f' = A_f \frac{E_f}{E_b} \quad \text{(1)}$$

The area of the composite section

$$A_c = A_f' + A_b \quad \text{(2)}$$

The moment of inertia of the transformed slab cross section

$$I_f' = I_f \frac{E_f}{E_b} \quad \text{(3)}$$

The distance between centroids of beam and slab

$$e_{bf} = y_{fb} + y_{bt} \quad \text{(4)}$$

The distances between centroids of composite section and slab and beam, respectively

$$e_{cf} = e_{bf} \frac{A_b}{A_c} \quad \text{(5)}$$

$$e_{cb} = e_{bf} \frac{A_f'}{A_c} \quad \text{(6)}$$

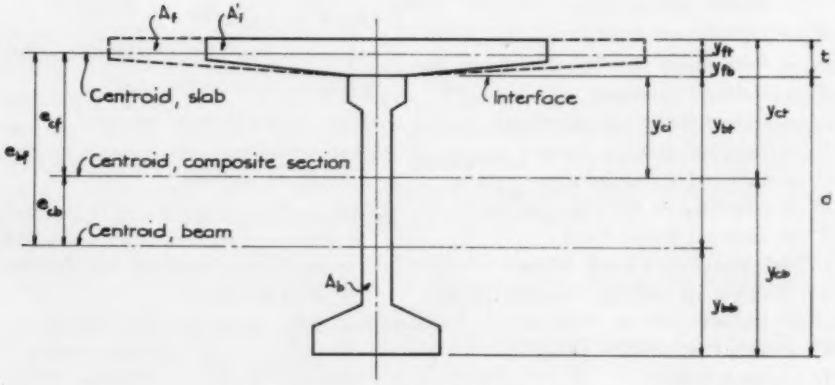


Fig. 2 — Geometry of section

The distances from centroid of composite section to fibers at top, interface, and bottom, respectively

$$y_{ct} = e_{cf} + y_{ft} \quad \text{.....(7)}$$

$$y_{ct} = e_{cf} - y_{fb} = y_{bt} - e_{cb} \quad \text{.....(8)}$$

$$y_{cb} = e_{cb} + y_{bb} \quad \text{.....(9)}$$

The moment of inertia of the composite section

$$I_c = I_f' + A_f' e_{cf}^2 + I_b + A_b e_{cb}^2 \quad \text{.....(10)}$$

FORCES AND STRESSES ON SLAB AND BEAM SECTIONS

The external force on slab, applied at centroid of slab, of a magnitude to cancel the differential shrinkage, is

$$P = \varepsilon A_f E_f \quad \text{.....(11)}$$

in which ε is the differential shrinkage, positive when slab shrinks more than beam. This is also the external compressive force on the composite section, applied at centroid of slab, which cancels the previously applied external tensile force (refer to Fig. 3).

If the compressive force P is shifted to the centroid of the composite section, Fig. 3(c), it is accompanied by a moment

$$M_c = P e_{cf} \quad \text{.....(12)}$$

The stresses on the cross section, Fig. 3(d), are determined in the usual manner, with allowance for the two load applications and con-

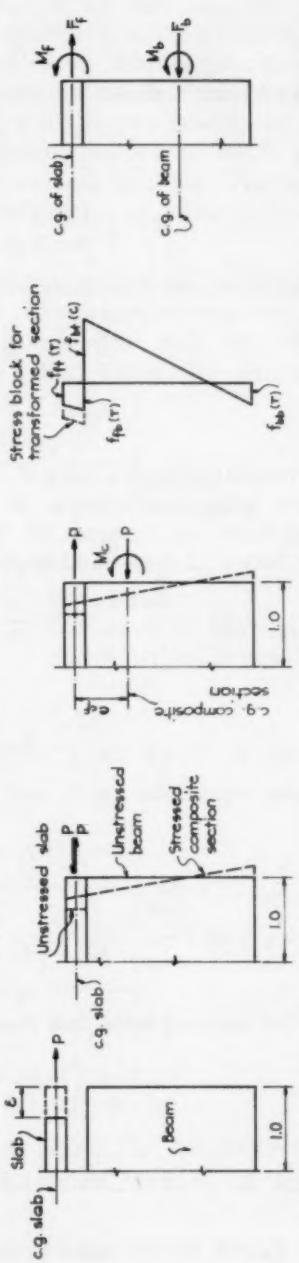

(a) - SLAB AND BEAM BEFORE INTEGRATION. STRESS STAGE 1.
Slab
Beam
c.g. slab
(b) - COMPOSITE SECTION AFTER EXTERNAL FORCE IS CANCELLED. STRESS STAGE 2.
Unstressed slab
Unstressed beam
Stressed composite section
c.g. slab
(c) - ALTERNATE FORCE APPLICATION. STRESS STAGE 2.
c.g. composite section
(d) - FINAL STRESSES ON TRUE COMPOSITE SECTION
Stress block for transformed section
(e) - INTERNAL FORCES AND MOMENTS ON SLAB AND BEAM
c.g. of slab
c.g. of beam

Fig. 3 — Unit length of span with externally applied forces and with internal stresses and forces

version from transformed to true section. The general equation is

$$f = \frac{P}{A} \pm \frac{My}{I}$$

The stresses in the slab, at top and interface fibers, respectively, are

$$f_{ft} = \left(\frac{P}{A_f'} - \frac{P}{A_c} - \frac{M_c y_{ct}}{I_c}\right)\frac{E_f}{E_c} \quad \text{(13)}$$

$$f_{fb} = \left(\frac{P}{A_f'} - \frac{P}{A_c} - \frac{M_c y_{cb}}{I_c}\right)\frac{E_f}{E_b} \quad \text{(14)}$$

The stresses in the beam, at interface and bottom fibers, respectively, are

$$f_{bt} = -\frac{P}{A_c} - \frac{M_c y_{ct}}{I_c} \quad \text{(15)}$$

$$f_{bb} = -\frac{P}{A_c} + \frac{M_c y_{cb}}{I_c} \quad \text{(16)}$$

The net resulting normal force F_f (positive when tensile) and moment M_f (positive when compression in top fiber) on the slab section are determined from the stresses in the slab [refer to Fig. 3(e)]. The stresses in terms of force and moment are

$$f_{ft} = \frac{F_f}{A_f} - \frac{M_f y_{ft}}{I_f}$$

and

$$f_{fb} = \frac{F_f}{A_f} + \frac{M_f y_{fb}}{I_f}$$

Solving these equations for F_f and M_f

$$F_f = \left(f_{ft} y_{fb} + f_{fb} y_{ft}\right)\frac{A_f}{t} \quad \text{(17)}$$

and

$$M_f = \left(f_{fb} - f_{ft}\right)\frac{I_f}{t} \quad \text{(18)}$$

Similarly, the normal force and moment in the beam are

$$F_b = \left(f_{bt} y_{bb} + f_{bb} y_{bt}\right)\frac{A_b}{d} \quad \text{(19)}$$

and

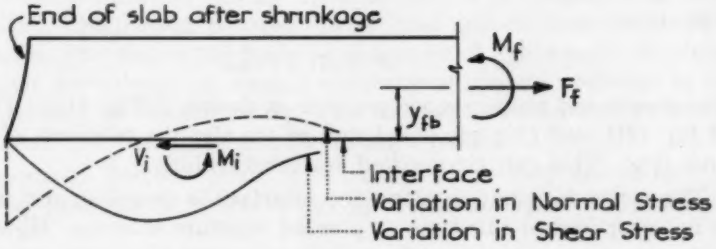
$$M_b = \left(f_{bb} - f_{bt}\right)\frac{I_b}{d} \quad \text{(20)}$$

The forces F_f and F_b are equal in magnitude but of opposite sign.

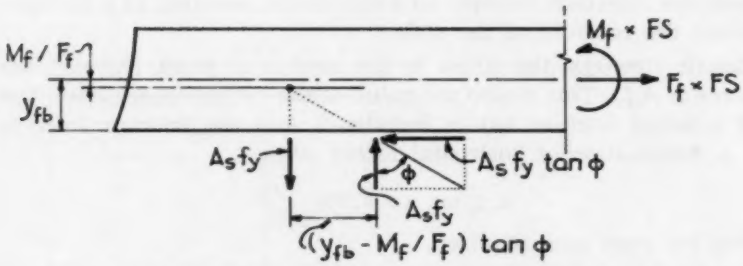
SHEAR AND MOMENT IN INTERFACE

Shear and normal stresses will exist in the interface, but only near each end of the span at a distance sufficient for the forces and stresses of the preceding paragraphs to be established. With ample reinforcement across the interface, this distance is in the order of half to full beam depth. In the remainder of the span there are no stresses in the interface, except for the secondary stresses required to bend the direct stresses along the deflected shape of the slab or beam. The assumption that a vertical plane section before bending remains plane after bending does not permit any other stress condition.

Near the end of the beam it is necessary to establish this condition. The shrinkage of the slab will cause shearing stresses in the interface, with tensile stresses in the adjacent fibers of the slab, and compressive stresses in the adjacent fibers of the beam. There will also be a tend-



(a) FORCES AND MOMENTS AT END OF BEAM



(b) ALTERNATE SETUP OF FORCES AT END OF SLAB TO DETERMINE REINFORCEMENT ACROSS INTERFACE

Fig. 4 — Force and moment in interface at end of slab

ency for the slab to curl away from the beam near the end, and this is resisted by tensile stresses across the interface. For static equilibrium there must be balancing compressive stresses farther along the interface. The resulting shear and moment, with qualitative stress distribution, are illustrated for the slab in Fig. 4(a).

Referring to Fig. 4(a), the shear and moment in the interface at the end of the slab are seen to be

$$V_i = F_f \quad (21)$$

and

$$M_i = -M_f + F_f y_{fb} \quad (22)$$

The shear and moment in the interface at the end of the beam can be obtained similarly:

$$V_i = -F_b \quad (23)$$

and

$$M_i = M_b + F_b y_{bi} \quad (24)$$

The shears and moments are positive as shown in Fig. 4(a). The shears of Eq. (21) and (23) are equal, and so are also the moments of Eq. (22) and (24). This can be verified by substitution.

The stress distribution along the interface is complex, and the writer is not prepared at this time to give an absolute solution. However, the following approach is simple and gives results that have proven satisfactory in the design of reinforcement for numerous composite beams.

Let it be assumed that resistance is provided by a force couple normal to the interface, and that the compressive part of the couple is accompanied by a friction or shear force in the plane of the interface. This is illustrated in Fig. 4(b). To satisfy equilibrium it is necessary that the forces from the interface combine to a horizontal resultant at a distance M_f/F_f below the centroid of the slab.

At ultimate strength, the stress in the steel is at yield (f_y) and the tensile force is $A_s f_y$. This is also the value of the compressive force. The angle of internal friction (ϕ) is mobilized, and the friction force is $A_s f_y \tan \phi$. Summation of horizontal forces gives

$$A_s f_y \tan \phi = V_i \, FS$$

or, solving for steel area required,

$$A_s = \frac{V_i \times FS}{f_y \tan \phi} \quad (25)$$

The same result could be obtained by summation of moments, or by analyzing the forces on the beam rather than the slab.

SLOPE AND DEFLECTION

Slopes and deflections are obtained by the usual integrations of the moment curve or by the conjugate beam method. The moment is constant through the length of the span, and

$$\text{Deflection at midspan} = \Delta_{max} = -\frac{M L^2}{8 EI} \quad \ldots\ldots (26)$$

In the above equation one substitutes either M_f, E_f, and I_f; M_b, E_b, and I_b; or M_c, E_b, and I_c for M, E, and I, respectively. Either set of substitutions will yield the same results since the slab, the beam and the composite slab-beam deflect the same amount at any common point.

EXAMPLE NO. 1

In December, 1957, the Concrete Technology Corp. of Tacoma made a full size test beam identical to beams that were later to be installed in the Norton Building in Seattle. The beam was of composite section, with precast, prestressed concrete beam and cast-in-place reinforced concrete slab. The slab in the building was to be of lightweight concrete, but practical considerations caused standard aggregate concrete to be used for the slab on the test beam. This slab was scaled down in width to compensate for the difference in Young's moduli.

Dimensions of the test beam with cast-in-place slab are shown in Fig. 5. The histories of concrete strengths and midspan deflection of

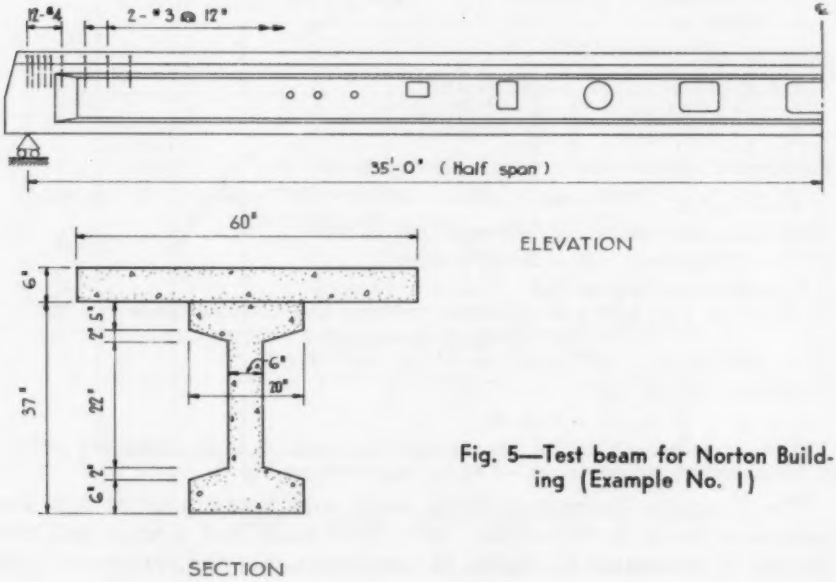


Fig. 5—Test beam for Norton Building (Example No. 1)

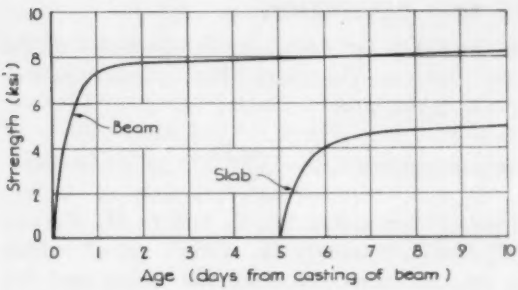


Fig. 6 — Concrete strengths (Example No. 1)

the beam are shown in Fig. 6 and 7, respectively. Given values and results of computations are as follows:

Concrete properties

$E_f = 1000$ ksi, based on judgement
$E_b = 7000$ ksi, derived from load-deflection measurements on beam, made prior to casting slab
$\varepsilon = 0.000450$ based on judgement

Properties of section

$A_f = 360$ sq in.; $A_f' = 51.43$ sq in.
$I_f = 1080$ in.^{4}; $I_f' = 154.29$ in.^{4}
$t = 6$ in.; $y_{ft} = y_{fb} = 3$ in.
$A_b = 404$ sq. in.; $I_b = 68{,}000$ in.^{4}
$d = 37$ in.; $y_{bt} = 18.9$ in.; $y_{bb} = 18.1$ in.
$A_c = 455.43$ sq in.
$e_{bf} = 21.9$ in.; $e_{cf} = 19.43$ in.; $e_{cb} = 2.47$ in.
$y_{ct} = 22.43$ in.; $y_{cb} = 16.43$ in.; $y_{cb} = 20.57$ in.
$I_c = 90{,}040$ in.^{4}

Fiber stresses

$P = 162.0$ kips; $M_c = 3148$ in.-kips
$f_{ft} = +0.2872$ ksi; $f_{fb} = +0.3171$ ksi
$f_{bt} = -0.9300$ ksi; $f_{bb} = +0.3634$ ksi

Forces and moments on slab and beam sections

$F_f = -F_b = +108.8$ kips; $M_f = +5.38$ in.-kips; $M_b = +2377$ in.-kips

Shear and moment in interface near end of span

$V_i = 108.8$ kips; $M_i = +322$ in.-kips
$f_y = 40$ ksi; $\phi = 60$ deg; $\tan \phi = \sqrt{3} = 1.73$
FS = 1.5 (the difference between cracking and ultimate loads is usually small, and the probability of overload due to shrinkage is negligible)
A_s (steel area) = 2.35 sq in.; use 12 - #4 bars (Fig. 5)

Deflection at midspan

$L = 70$ ft; $\Delta_{max} = -0.440$ in.
This computed deflection due to slab shrinkage is to be compared with an observed deflection of -0.48 in. as shown on Fig. 7.

The Concrete Technology Corp. made and tested another full size composite beam in September, 1959. This beam had a wide and thin flange, as illustrated in Fig. 8. It was hoped that the test would give

additional verification to the analysis presented in this paper, with regard to both deflection due to shrinkage and the early gripping of the beam by the slab. The verification was obtained, but it was not as clear cut as anticipated.

Young's modulus of the beam was determined by load-deflection tests before slab concrete was placed.

The placement of concrete in the slab started at one end and proceeded at a uniform rate across the length of the span, with the placement completed in 1½ hr. The first deflection readings were taken within half an hour after that, and were repeated at about 2-hr intervals. The first set of readings showed that the beam deflected as expected from the weight of the wet concrete, but subsequent readings showed that the beam moved upward. This is illustrated in Fig. 8. The upward movement peaked at 4½ hr, and from then on the movement was downward. It was thought that the heat of hydration in the slab was being transmitted to the top flange of the beam, thus causing it to increase in length and the beam to deflect upward. A thermometer placed on the slab and covered with an insulating blanket showed a substantial temperature rise. This beam differed from others that have been observed in that the top flange was unusually wide and thin, and

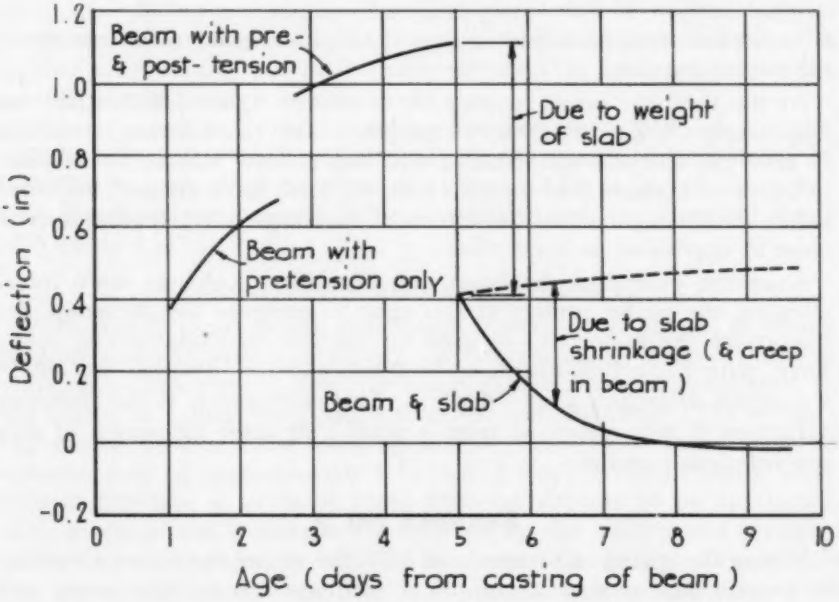


Fig. 7 — Observed deflection at midspan (Example No. 1)

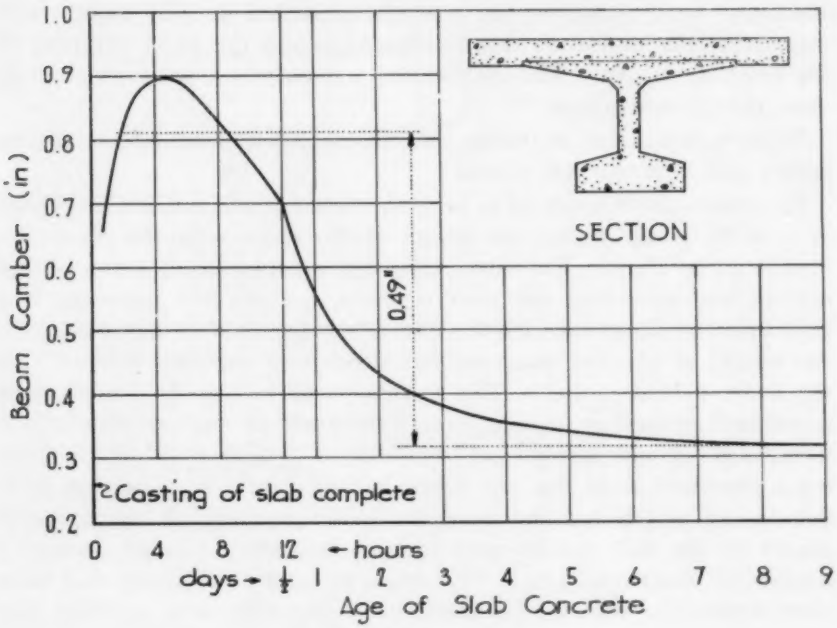


Fig. 8 — CTC test beam

it could therefore be expected that it would respond more readily to the heat in the slab.

At the time the beam reached its maximum upward deflection, the slab concrete had a compressive strength of 2000 psi, a Young's modulus of 2800 ksi, and the temperature rise had not yet started to subside. It therefore appears that the slab concrete must have gripped the beam before that time, and that the effects of shrinkage and temperature rise were in opposition to each other.

Since the effects of shrinkage and temperature change were intermingled, the writer prefers at this time to compute the shrinkage deflection on the same basis as used for the Norton Building test beam. Then, with $E_f = 1000$ ksi, $E_b = 6600$ ksi, and $\varepsilon = 0.000450$, we obtain a midspan deflection $\Delta = -0.489$ in. This corresponds to the observed deflection if it is measured from a point 2 hr after placement of slab concrete was complete.

EXAMPLE NO. 2

During the spring and summer of 1959, the writer examined a number of precast wall panels of composite section, with marble facing and concrete backing. The panels were cast absolutely flat, but developed

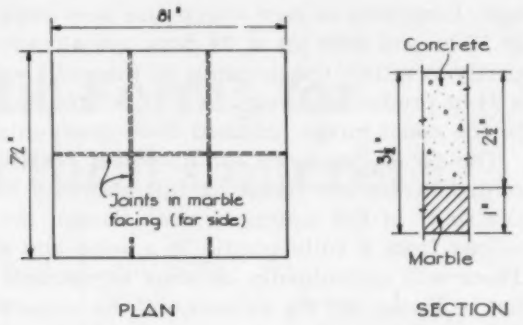


Fig. 9 — Wall panel with marble facing and concrete backing (Example No. 2)

a warp so that the surface took a slightly spherical shape when the concrete had cured. This was due to the restraining effect of the marble. It may be said that a truly flat panel can be obtained by grinding the marble facing to a plane surface after warping is complete, but such a solution was not considered to be practical. However, the warping was not excessive, ranging between ⅛ and 3/16 in. in a span length of 72 to 81 in.

Dimensions of a typical panel are shown in Fig. 9. The marble is attached to the concrete backing with stainless steel clips, but the warping characteristics show that full integration is obtained by bond, and this will be assumed in the analysis of the problem. The fact that the marble facing is made up of several pieces will be neglected, since the joint is very narrow. The concrete backing is reinforced with welded wire fabric, but its restraining effect as compared with the marble is small, and it is neglected.

With an effective Young's modulus for concrete of 1000 ksi, that for marble 6000 ksi, and the total free shrinkage of concrete equal to 0.000500, we compute deflections of 0.139 and 0.176 in. in the short and long directions, respectively. These compare with the observed range of 0.125 to 0.187 in.

COMMENTS AND CONCLUSIONS

With regard to predicting deflections caused by shrinkage, it is now possible to do so with an accuracy that is satisfactory from a practical viewpoint. However, it requires some experience and judgement in the choice of values for Young's modulus and shrinkage. The writer's experience verifies Ferguson's statement[2] that the effective Young's modulus may be approximately 1/40 that of steel. With the better qualities of concrete, a value of about 1000 ksi appears to be applicable. Both modulus and shrinkage are affected by the quality and quantity of the ingredients that go into the concrete, and depend also on how the concrete is placed and cured. More research is indicated, with emphasis on the effective modulus and the effective differential shrink-

age. Concretes of zero slump and very high strength, such as 5000 psi at 12 hr and 9000 psi at 28 days, are already being used in production, partly to satisfy the demands on the end product and partly to facilitate a 24-hr production cycle. Such concretes should be included, in addition to the usual mixes obtained from ready-mixed concrete plants.

The forces, moments, and stresses resulting from the analysis presented herein are based on full elasticity (with reduced E_c value for the slab) of the materials, even though the cast-in-place concrete develops from a fully plastic to a solid and essentially elastic material. There will undoubtedly be some adjustment in stresses, and they may not be linear, but the existence of the moments and forces in the magnitudes indicated (long-term creep neglected) is well established by the deflection characteristics of the well-seasoned precast beam.

The stress distribution in the interface near the end of the span is still uncertain. Even though the approach given for the determination of reinforcement has served satisfactorily, a better understanding and possibly a better approach may result from more research in this field.

The method of this paper may be applied to other problems of a similar nature, such as a cast-in-place concrete slab on a steel beam, and a slab-beam combination with a temperature difference between slab and beam.

ACKNOWLEDGMENTS

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Title No. 56-57

Plastic Forms for Architectural Concrete

By J. A. HANSON

Pleasing and unusual architectural effects in concrete are being obtained by the use of forms made of plastics. The architectural treatment may consist of both surface finish and bold pattern decoration. The concrete surfaces can be either glossy-smooth or textured. In either case, the surface is free of open air and water pockets. No special facing mixes, no parting agents, and no polishing or grinding are required. The extremely fine smooth finish of plastic-formed concrete greatly improves the attractiveness of integral colors, and due to the high reflectivity, smaller amounts of pigment are required to obtain a given color intensity.

A considerable number of reuses are obtainable with the plastic forms, thus reducing the cost through multiple use. Laboratory tests established optimum curing conditions for this type of architectural concrete. It was found that temperature control is important if a high degree of reflectivity is desired.

Although the present development of plastic forms was initiated only 3 years ago, a considerable number of applications have been made in the field. Several of these are described.

■ CONCRETE IS RECOGNIZED AS A VERSATILE building material, but the architectural beauty that can be achieved through different textures, patterns, and colors is not so well known. The recent development of casting concrete against plastic forms to produce satin-smooth glossy surfaces is one more method that can be added to the variety of surface treatments available for producing attractive concrete panels. This versatility is demonstrated by Fig. 1, exhibiting concrete panels with bold patterns and color combinations that can be readily seen and appreciated at distances of several hundred feet, and showing also how the depth of relief and composition of the decorations create contrasting shadow patterns. The lower design is particularly effective in producing a constantly changing appearance as the direction of the sunlight changes throughout the day.

These panels were photographed exactly as they were obtained on stripping of the plastic form. The surfaces are free of open air and water pockets and no surface treatments such as special facing mixes, polishing, grinding, or waxing were employed. Also, no parting agent was

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applied to the form prior to casting. The concrete as cast was an ordinary mix originally designed for structural purposes. As such, the mix contains ¾-in. maximum size aggregate. The reader will appreciate that the black and white photographs shown here do not do justice to the lustre and color of the actual concrete surfaces.

While the "gloss" of the plastic formed concrete has been the focus of attention during the past 3 years of development, pleasing surface textures are also obtainable by the same method of casting. This texture may be imparted to the plastic as produced at the factory, or special textures may be formed for particular jobs. The plastic sheet most widely used by the industry for concrete molding purposes is formed with high gloss on one side and leather-like (hair-cell) texture on the other, thus giving the user a convenient choice. The textured surfaces may be preferred by many architects, especially under exposure conditions that might be considered detrimental to the permanence of gloss. A basic advantage of casting concrete against plastic is the as-

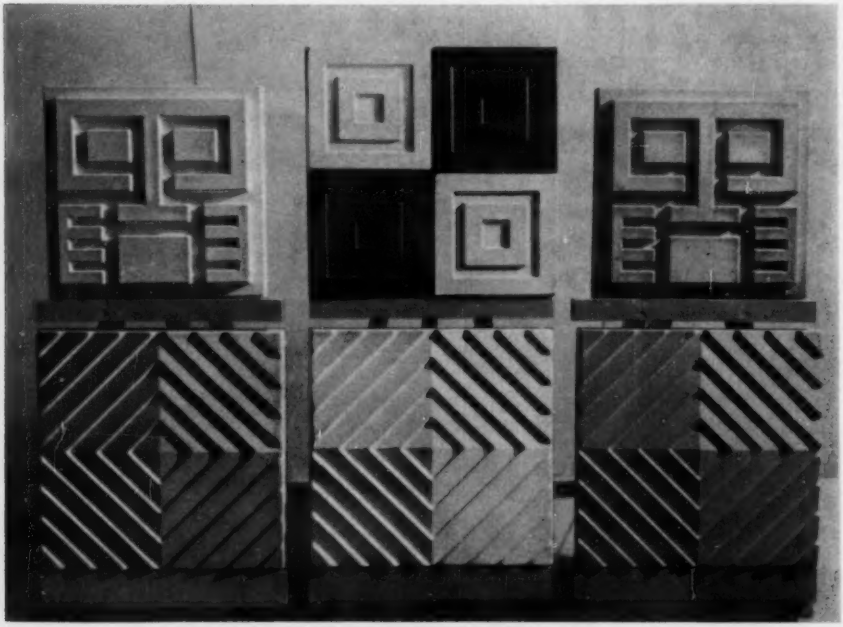

Fig. 1—Bold designs and shadow patterns

surance of uniform color and appearance of the concrete, particularly since the form has negligible absorption, and discoloration by parting agents is eliminated. Several companies producing plastics are introducing thin stock that can be produced in long rolls for casting flat concrete surfaces where such uniformity is desired without distinctive pattern.

PLASTIC FORMS

The 21 x 21-in. plastic form shown in Fig. 2 was used to mold the upper concrete panel. This form was molded from sheet plastic approximately 1/16 in. thick. Pattern decoration such as this can be imparted to the thermoplastic sheet either by positive pressure or vacuum forming. Vacuum forming of the plastic may offer considerable economic advantage in that high finish quality is not required in the original model over which the plastic is formed. In this process the plastic comes in contact with the original pattern on the side opposite to that against which concrete will be cast, and minor imperfections in the pattern are not reproduced in the concrete.

The pattern may be of various materials such as wood, plaster of Paris, metal, or glass. Small diameter holes are drilled through the model at strategic low points in the design. These holes are incorporated in a vacuum system, usually by placing the model in a pan that has a marginal lip for automatically sealing the edges of the plastic sheet. After this sheet has been heated above 300 F, it is placed over the pan, and the vacuum valve is opened. Due to the extreme flexibility of the plastic at this temperature, almost instantaneous forming over the model then occurs. Most of the firms making plastic forms for concrete are existing jobbers in the plastics industry. However, in at least one instance, a concrete producer has constructed his own vacuum forming equipment.

Fig. 2—Plastic form and concrete panel

The construction of the model should incorporate sufficient draft of the pattern lines to insure ease of stripping of the concrete. Depth of relief should be considered in relation to width of pattern valleys to avoid over-thinning of the plastic material. Precise tolerances of large finished concrete products may require allowance for the cooling shrinkage of the plastic. The model dimensions of small refine-

ments and niceties of design must provide proper allowance for plastic thickness if they are to be faithfully reproduced. The plastic materials that have been used to form concrete range from a few mils thickness to ⅛ in. or greater. The extremely thin sheets offer difficulty in the prevention of wrinkling and are subject to accidental puncturing and tearing.

Size of the individual designs may range from 1 sq ft or smaller up to approximately 4 ft wide by 7 ft long. This length limitation may be exceeded with the use of the thin "roll stock" being introduced by plastic producers. However, for the present the width is limited to 4 ft by the machinery producing the plastic sheet stock at the factory.

The requirements for a successful plastic-formed concrete are similar to those for all types of high quality architectural concrete. The concrete mix should be well proportioned and controlled. At least 6 sacks of cement per cu yd of concrete are recommended and aggregate should be well graded with sufficient fines to assure a good trowel finish. Maximum size of aggregate should be consistent with the thickness of the slab. Slump, when using lightweight aggregate, should not exceed 3½ in., and for normal-weight aggregate concrete the slump should not exceed 4½ to 5 in. Air entrainment is recommended for increased durability and workability. The form liner should be used clean and dry. The liner surface can be highly polished by rubbing with a dry towel, and this polish will be imparted to the concrete. No parting agents should be employed since these agents remove the sheen and give a smeared appearance to the cured concrete. Thorough vibration is necessary to break air bubbles from the formed surface, but, in accordance with sound concreting practice, overvibration is to be avoided.

If a high degree of gloss is desired, the best results are obtained when the concrete is cured under some type of waterproof membrane, such as polyethylene film, preserving the original moisture in the mix without gain or loss of water. Specimens cured under excess moisture conditions have practically no gloss. Specimens which were allowed to dry during the curing period gave inferior results compared to the membrane curing. It was also found that temperature of curing must be maintained close to that of casting to obtain high reflectivity. The temperature coefficient of expansion of plastics is much higher than the coefficient for concrete, and thus separation of the plastic from the concrete will occur with a considerable temperature change. For the highest degree of gloss the temperature change between casting and stripping should not exceed ± 5 F, although these limits may be extended for many applications. A dull but smooth surface results when the plastic is separated from the concrete during curing, whether this separation is due to temeprature change, accidental rough handling, or deliberate early stripping. For high gloss, curing in the plastic form for at least

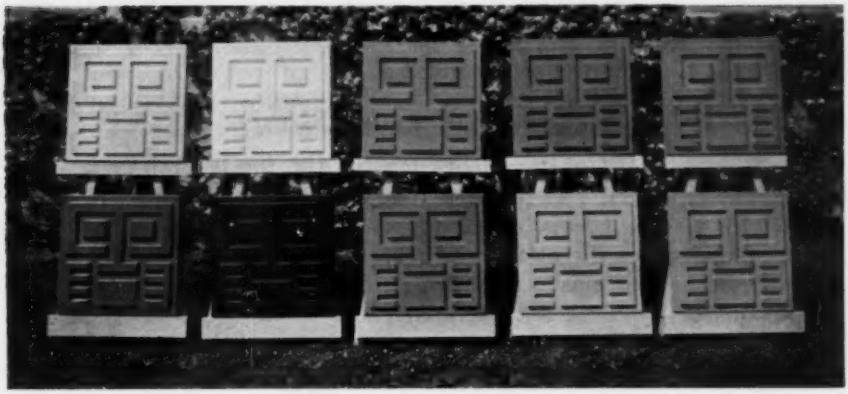

Fig. 3—These ten panels were cast with structural grade lightweight concrete mixes using white cement, with the exception of the gray panel which contained gray cement without pigment. Pigments were used in amounts of from 1 to 6 percent by weight of cement to produce the desired shades

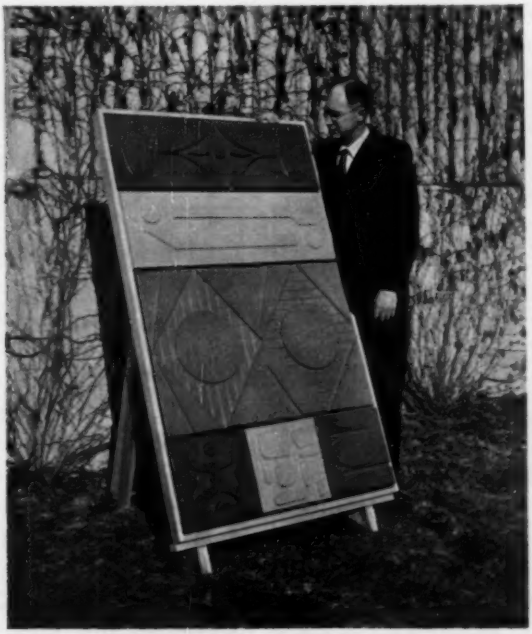

Fig. 4—Six concrete panels in brilliant colors with extreme glossiness. Some of the patterns are quite intricate with the concrete formed in several planes

48 hr and preferably 3 days is recommended and some improvement of gloss has been noted with longer periods. Curing conditions are much less critical if a textured plastic is used.

LABORATORY DEVELOPMENT

A variety of plastic formulations was tried in the laboratory as forming materials, and it was found that many plastics will give a smooth finish to concrete. However, factors other than surface finish must be considered, such as the number of reuses which may be obtained from a single piece. The plastic must have sufficient flexibility to withstand the strains of handling and stripping, and at the same time, it must be rigid enough to mold architectural patterns without wrinkling or sagging in unsupported locations. The most satisfactory materials we have used are various butadiene styrenes and linear polyethylenes. The initial development work was carried out in cooperation with the U. S. Rubber Co. After trials of several existing formulations an improved product was found that had sufficient reuseability to be considered economical for architectural concrete.

One of the questions most frequently asked in regard to this type of architectural concrete is that of durability. In laboratory tests which are still in progress, samples of the glossy concrete have been subjected to 150 freezing and thawing cycles and to 300 cycles of wetting and drying. No significant surface deterioration or discoloration has been found. In spite of this satisfactory behavior under laboratory testing, experience may indicate that textured surfaces are to be preferred to glossy surfaces for exterior exposure. More information is needed before questions regarding durability of the surface can be answered with complete assurance.

COLORED CONCRETE

One of the pleasant surprises connected with the plastic forms has been the enhanced appearance of colored concrete. With the much improved surface, the brilliance and uniformity of colors has been raised to a high degree compared to colors attained with other types of forming or finishing. The ten panels of Fig. 3 were cast with structural grade lightweight concrete mixes using white cement, with the exception of the gray panel which contained gray cement without pigment. Except for the blue and the black, the pigments were commercial synthetic iron oxides. The pigments were preblended dry with the white cement prior to mixing. With the increased brilliance of the colors, smaller amounts of pigment were necessary than are normally recommended. In these panels the pigments were used in amounts of from 1 to 6 percent by weight of cement to produce the desired shades.

The panels of Fig. 4 are similar to those of Fig. 3 but the reflectivity of the concrete was increased to a high degree with the use of improved plastic sheet. With this extreme glossiness, the red, yellow, green, and slate gray colors were obtained in radiant shades. Some of the patterns in these six panels are quite intricate with the plastic being vacuum formed into several planes. In addition, many of the decorative lines had very narrow widths relative to their depth. In spite of these difficulties, the original wood patterns were faithfully reproduced in the plastic.

FIELD USAGE

Although the development of casting concrete in plastic forms has been proceeding for only some 3 years, this method has already been used on a number of structures.

In the 88-100 University Avenue Office Building, Toronto, Ont., Canada, the method has been used for pan ceilings. In these jobs, the usual metal pans were replaced with fiber-reinforced plastic pans. In other respects the concreting and form removal procedure was similar to that of usual waffle grid ceilings. The use of plastic pans for waffle construction requires only minor cleanup and permits exposed ceilings if so desired.

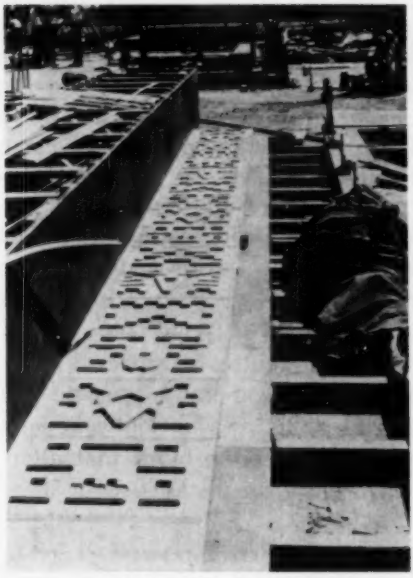

Fig. 5 (left)—Plastic forms in position for casting column of PCA Fire Research Center. Fig. 6 (right)—Details of completed column

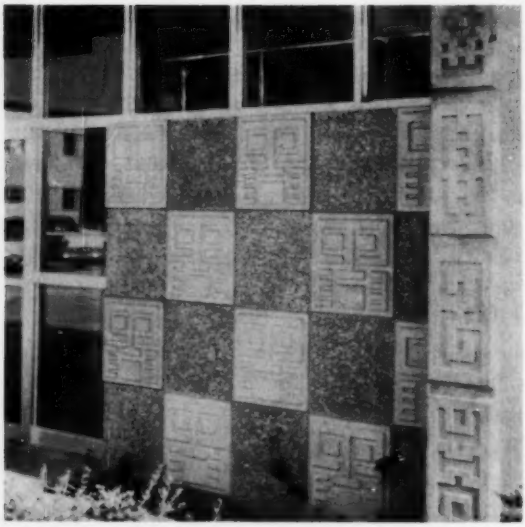

Fig. 7—Decorative panel at entrance of PCA Fire Research Center

The new Fire Research Center of the Portland Cement Association used plastic forms in several instances. This building was precast on the site and erected into place. The design features 11 columns on each side that were cast face downward against plastic forms. Fig. 5 shows the installation of the plastic in the bottom of the column prior to placing the second side wall. Seventeen pieces of plastic, each 2 ft square, were required for each column. These pieces had flat surfaces at all edges and were held in place by the chamfer strips along the sides, and abutting edges of the plastic forms were covered with a tapered wood divider strip. Fig. 6 presents a detailed view of the clean appearance of the completed column after erection. Six different patterns are used in a predetermined sequence over the entire building. The architect preferred to avoid glossiness of the concrete and specified the leather-like texture for the plastic surface. In these designs the pattern is ¾ in. deep. Minimum internal vibration was used in placing the concrete, but the column surfaces are essentially void free.

The decorative panel at the Fire Research Center entrance (Fig. 7) effectively used a checkerboard combination of exposed aggregate and plastic-formed concrete. Each panel is 21 x 21 in. The lobby beyond this entrance is lined with 12 x 12-in. glossy concrete block that alternate flat and patterned surfaces. These blocks were installed with ⅜ in. flush mortar joints.

Construction firms in Milwaukee have used plastic-formed concrete for the window opening decoration of several apartment and office buildings. The straight lines of one of these designs, shown in Fig. 8,

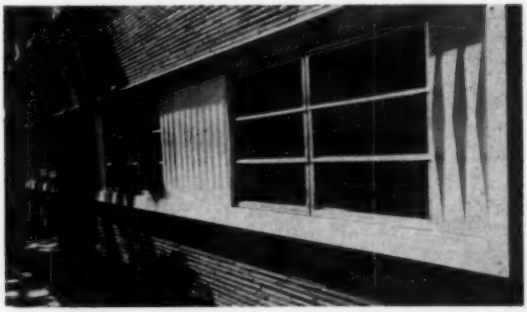

Fig. 8—Decoration of office building in Milwaukee

are simple but effective. A somewhat more spectacular design of interlocked pyramids was also utilized to decorate larger areas of window opening decoration.

In connection with their new office building the Riverside Cement Co. in California decorated a 160 ft long fascia beam (Fig. 9) with a design similar to that employed in folded roofs. The feature of this job is that the plastic forms a cast-in-place vertical surface. It is considerably more difficult to obtain void-free surfaces on vertical than on horizontal planes. On this job great care was exercised to secure complete vibration of the concrete that was directly in contact with the plastic. The architect wisely employed nearly vertical lines in the pattern, with no horizontal projections of the plastic to entrap air and water bubbles as they tend to rise during consolidation. Near the top of the flutes there is one point at which air would certainly be entrapped in ordinary cir-

Fig. 9—Decoration of cast-in-place fascia beam, Riverside Cement Co. office building

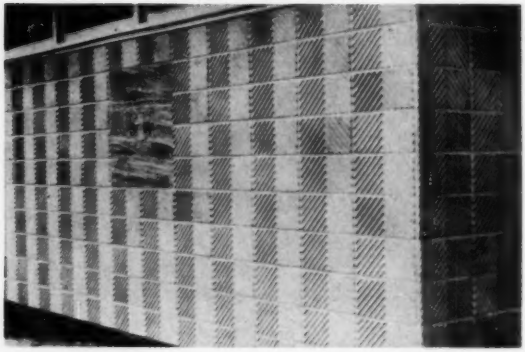

Fig. 10—Plastic-formed masonry facing, Elk's Lodge, Chaffee, Mo.

cumstances. Here the contractor drilled ⅛-in. holes in the plastic and found that this effectively removed entrapped air.

Fig. 10 shows how plastic forms were used at the entrance to the Elk's Club at Chaffee, Mo. In this installation commercial 8 x 16-in. concrete block were faced with a cement-sand mixture cast in a plastic pan. The bond between the facing and precast block depends only on the natural bond of the two concretes. After facing in a precasting plant, the wall was laid up as a stacked bond with standard masonry

Fig. 11—Largest plastic-formed job to date: Bullock's Department Store, Santa Ana, Calif.

procedures. The effective use of shadow patterns gives this wall the appearance of a stacked system of 8 x 8-in. masonry.

Bullock's Department Store building in Santa Ana, Calif., represents one of the largest uses of precast plastic-formed concrete to date. Fig. 11 shows a view of one side of this attractive structure. Approximately 23,000 sq ft of precast elements were used in the form of 1-ft squares. Each small panel was 1 in. thick. The panel fabricator reports that 1000 pieces of plastic were used; thus, the average reuse of each piece of plastic was 23 times. The architect specified a smooth but nonreflective surface and consequently the forms were stripped within a few hours after casting. The specifications also required that the concrete panels be colored with three shades of beige placed randomly over the structure. The use of these three colors is particularly apparent in the heavily shadowed portion of the figure. The forms were molded with square edges on two sides and the other sides had some draft to allow stripping. The 1-sq ft units were then attached to the back-up wall with parge mortar in a series of four to form a larger design, 2 ft to the side. Interior joints were kept tight by pushing the no-draft sides together, and exterior joints of each 2-ft square were spaced uniformly with a removable ¼-in. rope.

The use of concrete screens for combined functional and architectural purposes has been growing rapidly. Grillework of this type can add decoration to structures while providing such functions as ventilation, equipment cover, or sunlight shielding. Concrete screens placed over the large window areas common to many office buildings will materially reduce air conditioning cost. Widespread use in the past has been somewhat hampered by the intricate form necessary to cast these products in concrete. Vacuum forming of plastic is a good solution to this problem and was employed to cast the screens shown in Fig. 12. These buildings

Fig. 12—Concrete screens providing mechanical equipment cover

are located in Bullock's shopping center in Santa Ana. The screens which provide shielding of mechanical equipment located on the roof, are anchored both mechanically and with mortar. A pebble texture was applied to the concrete surface by the plastic.

CONCLUSION

There will be an ever increasing use of plastic forms as architects and concrete fabricators become familiar and adept in their use. Complex designs and brilliant colors, as indicated by Fig. 4, can be obtained economically. The architect's creativeness is given a wide range of formal expression, and his designs can be obtained economically. The outstanding advantages offered by plastic forms are ease of stripping, absence of air and water voids, uniformity of natural appearance, patterns visible from greater distance, beauty of texture and design, and realization of the full potential inherent in permanent color decoration. These characteristics give promise that plastic forms will become one of the major methods of achieving architecturally pleasing concrete surfaces.

ARCHITECTURAL CREDITS

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2. Fire Research Center, Portland Cement Association, Skokie, Ill.—Dunlap & Esgar, Architects.
3. Office building in Milwaukee, Wis.—Megal Construction Co.
4. Riverside Cement Co. office building, Crestmore, Calif.—Allison & Rible, Architects.
5. Elk's Club, Chaffee, Mo.—Fred Dormeyer, Jr., Architect.
6. Bullock's Department Store, Santa Ana, Calif.—Charles Luckman Associates, William L. Pereira, Associate Architect.

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Title No. 56-58

Final report of a project sponsored by Reinforced Concrete Research Council at the University of Illinois

Effect of Floor Concrete Strength on Column Strength

By ALBERT C. BIANCHINI, ROBERT E. WOODS, and CLYDE E. KESLER

Forty-five specimens representing portions of the corner, edge, and interior column and floor sections of a typical structure were tested under axial compressive loads and the results analyzed to determine the following: (1) how large a differential in column concrete strength and floor concrete strength could be tolerated without decreasing the load-carrying capacity of the column, and (2) the allowable load-carrying capacity of the column if this differential is exceeded. The following variables were included: type of specimen, column concrete strength, and floor concrete strength.

From the analysis of the test results, a procedure was developed for computing the ultimate load of a column in which the column concrete is intersected by floor concrete. These limited tests indicated that the column strength is a function of the ratio of column concrete strength to floor concrete strength and the number of restrained edges tributary to the column. No reduction in column strength occurred for ratios of column concrete strength to floor concrete strength up to 1.4 for all types of specimens and up to 1.5 for most types of specimens.

■ IN PRESENT DAY REINFORCED CONCRETE CONSTRUCTION, substantial economies may be achieved by designing the floors, which may consist of slabs or slabs and beams, with medium strength concrete and the columns with high strength concrete. In the resulting structure, layers of floor concrete intersect the columns at each floor level. As these layers are usually made of lower strength concrete than the column itself, it is probable that under some circumstances such layers may decrease the load resisting capacity of the column.

The effect of the floor concrete on the strength of a column may be expected to depend on the lateral restraint offered to the lower strength floor concrete, on the relative strengths of the two concretes, on the relative thickness of the floor and the size of the column, on the percentage of column reinforcement, and on the eccentricity of the load.

Albert C. Bianchini, member of ACI Committee 208, is an assistant professor of theoretical and applied mechanics at the University of Illinois, Urbana. In addition to teaching courses in mechanics, Professor Bianchini has been actively engaged in work on various research projects in the field of reinforced concrete dealing with welded wire fabric, shear behavior, bond, and cracking. ● ACI member **Robert E. Woods** is a former research assistant in the Department of Theoretical and Applied Mechanics, University of of Illinois. Mr. Woods is currently employed by the bridge design section of the State Highway Department of Indiana. ● **Clyde E. Kesler**, professor of theoretical and applied mechanics, University of Illinois, has published numerous papers describing research in shear, fatigue, creep, and sonic behavior of concrete. An ACI member since 1947, he serves on the Technical Activities Committee and several committees including 209, Volume Change and Plastic Flow; 213, Properties of Lightweight Aggregates and Lightweight Aggregate Concrete; 215, Fatigue of Concrete; and ACI-ASCE Committee 326, Shear and Diagonal Tension.

The restraint to the lower strength concrete may be offered by the surrounding floor, by the floor reinforcement, and by the column concrete located above and below the floor layer. The restraint may entirely prevent failure of the floor concrete, as may well be the case for interior columns in which the surrounding floor provides a very effective lateral restraint. In other cases, however, the restraint may just raise the strength of the layer of floor concrete above the value expected on the basis of known cylinder strength. The restraint offered by the floor is likely to be different for each of the three types of columns: corner, edge, and interior. The restraint offered by the floor reinforcement will probably vary with the amount and arrangement of this steel. And the restraint offered by the column concrete may be expected to be a function of the ratio of floor thickness to column size and of the ratio of concrete strengths.

The effects of eccentricity of loads are related to the state of stress in the column-floor joints. Column-floor joints loaded eccentrically, in which the weaker floor concrete is only restrained laterally around a portion of its perimeter, may carry larger loads than concentrically loaded column-floor joints provided the eccentricity is in the right direction.

Object and scope

The object of this investigation was to determine how large a differential in column concrete strength and floor (slab or slab and beam) concrete strength could be tolerated without decreasing the load-carrying capacity of a column, and to determine the allowable load-carrying capacity of the column if this differential was exceeded.

The investigation included 54 axially loaded column specimens. Of the 54 specimens, 45 represented portions of the corner, edge, and interior column and floor sections of a typical structure and each specimen consisted of two tied columns with an intersecting floor between the two columns. The remaining nine specimens consisted of plain tied columns without any intersecting floor concrete. For the column and floor specimens, the ratio of column concrete strength to floor concrete

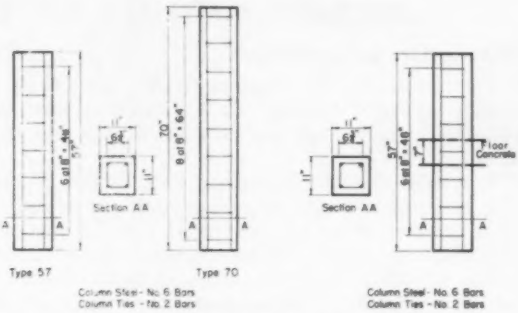

Fig. 1 (left) — Details of column specimens (column series). Fig. 2 (right)—Details of sandwich column specimens, Type S (slab series)

strength was varied systematically while the floor thickness, column cross section, and the percentage of column reinforcement were held constant.

Outline of tests

The investigation included seven different types of specimens and was divided into three series: column series, slab series, and beam series. An outline of the tests given in Table 1 indicates that the major variables were: type of specimen, column concrete strength, and floor concrete strength. The types of specimens investigated are shown in Fig. 1 through 7. In all specimens an axial compressive load was applied to the columns.

Notations

The following notations are used:

A_c = net area of concrete in column section

A_g = gross area of column section

A_s = area of column reinforcement

C = ratio of the concrete stress on an axially loaded column without floor (slab or slab and beam) concrete to the compressive strength of a 6 x 12-in. concrete cylinder

f_c' = 28-day standard cylinder compressive strength

f'_{cc} = 27-day, 28-day, or 29-day compressive strength of 6 x 12-in. *column* concrete control cylinders

f'_{cf} = 28-day compressive strength of 6 x 12-in. floor (slab or slab and beam) concrete control cylinders

f'_{cp} = apparent strength of floor concrete

f_s = 0.40 f_y, nominal allowable stress in the column reinforcement

f_y = yield point stress of column reinforcement

G = geometry of specimen

P = ultimate strength of a concentrically loaded column

P_a = total allowable axial load on the column

P_{calc} = $A_s f_y + CA_c f'_{cp}$. Calculated load on the specimen where $C = 0.85$

P_{test} = maximum test load on specimen

p_c = percentage of column reinforcement

p_g = ratio of the cross-sectional area of column reinforcement to the gross area A_g

p_f = percentage of floor reinforcement

R_c = restraint afforded the floor portion of the column section by higher strength column concrete

R_l = lateral restraint afforded the floor portion of the column section by overhanging floor concrete

SPECIMENS, MATERIALS, AND TEST PROCEDURE

Description of specimens

The details and dimensions of the specimens used in this investigation are shown in Fig. 1 through 7. All columns used in this investigation were tied columns 11 in. square and reinforced with four #6 bars. Column ties consisted of #2 bars.

The details of the specimens used in the column series are shown in Fig. 1. The specimens in this series had no intersecting floor concrete; they were either 57 in. long or 70 in. long and were used to evaluate the adequacy of the present ACI Building Code (318-56) ultimate load column formula.

The details of the four different types of specimens tested in the slab series are shown in Fig. 2 through 5. The specimens in this series consisted of two square tied columns and a slab 7 in. thick placed between the columns. The slab portions were reinforced on top and bottom with #4 bars as shown in Fig. 3 through 5. In the sandwich column specimens, Type S, the slab was cut off along the column periphery. For the corner column specimens, Type C, the slab was square and the column was in one corner. In the edge column specimens, Type E, the slab was rectangular and the column was in the middle of one side. For the interior column specimens, Type I, the slab was square and placed concentrically with respect to the column.

Details of the two different types of specimens tested in the beam series are shown in Fig. 6 and 7. The specimens in this series consisted of two square tied column sections and a 7 in. thick slab and 8 x 20-in. beams placed between the columns. The slabs were reinforced on top and bottom with #4 bars, and the beams reinforced with #8 bars. In the beam sections #3 bars were used for the stirrups. In this series, Type E and Type I specimens were similar to the Type E and Type I specimens of the slab series. In the edge column specimens,

TABLE 1—OUTLINE OF TESTS

Column series		Slab series				Beam series			
f'_{cc}, psi	Specimen No.*	f'_{cc} psi	f'_{cf} psi	$\frac{f'_{cc}}{f'_{cf}}$	Specimen No.†	f'_{cc}, psi	f'_{cf}, psi	$\frac{f'_{cc}}{f'_{cf}}$	Specimen No.†
Series 57		Type S				Type E			
7500	C7557	7500	2500	3.0	S75S3.0	9000	3000	3.0	B90E3.0
6000	C6057	6000	2500	2.4	S60S2.4	7500	2500	3.0	B75E3.0
5000	C5057	5000	2500	2.0	S50S2.0	6000	2000	3.0	B60E3.0
4500	C4557	3750	2500	1.5	S37S1.5				
3750	C3757	Type C,E,I				Type E,I			
3000	C3057	9000	3000	3.0	S90C,E, or I3.0	6000	3000	2.0	B60E or I2.0
Series 70		7500	2500	3.0	S75C,E, or I3.0	5000	2500	2.0	B50E or I2.0
7500	C7570	6000	2000	3.0	S60C,E, or I3.0	4000	2000	2.0	B40E or I2.0
5000	C5070	Type C,E,I				Type E			
3000	C3070	6000	3000	2.0	S60C,E, or I2.0	4500	3000	1.5	B45E1.5
		5000	2500	2.0	S60C,E, or I2.0	3750	2500	1.5	B37E1.5
		4000	2000	2.0	S40C,E, or I2.0	3000	2000	1.5	B30E1.5
		Type C,E,I							
		4500	3000	1.5	S45C,E, or I1.5				
		3750	2500	1.5	S37C,E, or I1.5				
		3000	2000	1.5	S30C,E, or I1.5				

*The designation of each specimen in the column series consists of one letter and four numbers. The first letter C designates the column series; the next two numbers represent the nominal column concrete strength (75 for 7500 psi, 50 for 5000 psi, etc.); and the last two numbers represent the height of the column (57 for 57 in., 70 for 70 in.).

†The designation of each specimen in the slab and beam series consists of two letters and four numbers. The first letter designates the phase of the investigation (S—slab series, column to slab; B—beam series, column to slab and beam); the next two numbers represent the nominal column concrete strength (75 for 7500 psi, 50 for 5000 psi, etc.); the second letter stands for the type of specimen (S—sandwich column, C—corner column, E—edge column, I—interior column); and the last two numbers represent the ratio of the nominal column concrete strength to the nominal floor concrete strength.

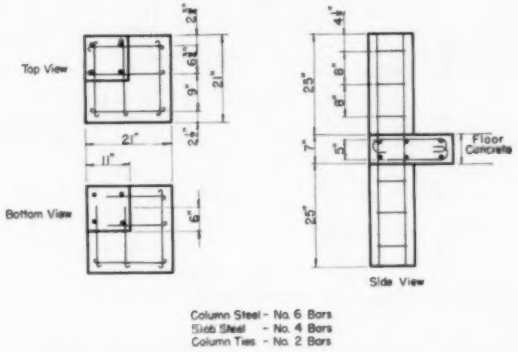

Fig. 3 — Details of corner column specimens, Type C (slab series)

Type E, the slab was rectangular and the column was in the middle of one side. Also, two beams were placed underneath, parallel and flush to the outside edge of the slab and connected into the column. For the interior column specimens, Type I, the slab was square and placed concentrically with respect to the column. Also, beams extended in four directions underneath the slab from the column faces.

Materials

Cement — A standard brand of Type I portland cement was used throughout the investigation.

Aggregates — Both the well graded fine and coarse aggregates used in the concrete mixtures were from the Wabash River near Covington, Ind. The sand had a fineness modulus of 3.12 and the gravel had a fineness modulus of 6.75. The bulk specific gravities, based on saturated surface dry, were approximately 2.65 and 2.70 for the sand and gravel, respectively. The absorption of both fine and coarse aggregate was about 1 percent, by weight, of the surface dry aggregate.

Concrete mixtures — The investigation involved ten different concrete strengths ranging from 2000 to 9000 psi. Mixes were designed for 28-day nominal compressive strengths.

The average column and floor compressive strengths, f'_{cc} and f'_{cf}, determined from a minimum of five 6 x 12-in. control cylinders, are listed in Tables 2 and 3 for the specimens in the column series and the slab and beam series, respectively.

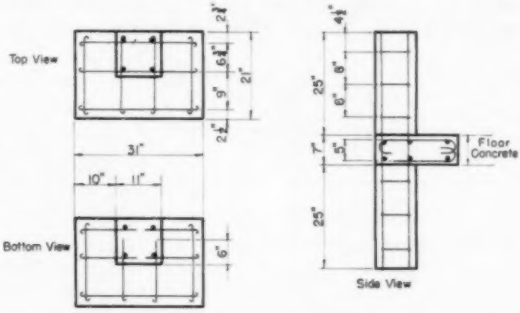

Fig. 4—Details of edge column specimens, Type E (slab series)

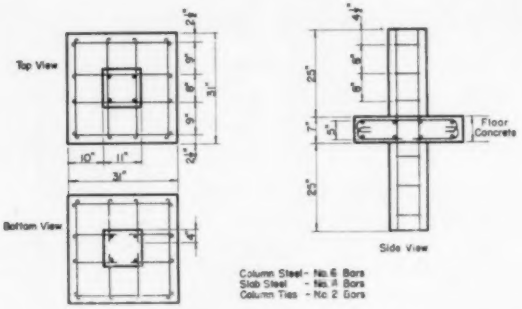

Fig. 5 — Details of interior column specimens, Type I (slab series)

The compressive strength values listed in Table 2, for the bottom column and top column of each specimen in the column series are 28-day strengths. In addition, the average of these two strengths is given. In Table 3, the compressive strength values listed for the bottom column, top column, and intersecting floor of each specimen in the slab series and beam series are 29-day, 27-day, and 28-day strengths, respectively.

Reinforcement — Intermediate grade deformed bars, satisfying the requirements of ASTM A 305-53T and A 15-54T, were used as reinforcement. The column reinforcement consisted of four #6 bars with #2 bars as ties. The slabs were reinforced, both on top and bottom, with #4 bars and the beam reinforcement consisted of four #8 bars. Beam stirrups were #3 bars.

In Tables 2 and 3, the yield point and ultimate strength values of the column reinforcement are listed for each specimen tested. The column reinforcement for any one specimen was cut from one bar 30 ft long. One coupon 2 ft long was then cut from each bar and tested. A typical tensile stress-strain curve for the column reinforcement is shown in Fig. 8.

Fabrication and curing

All specimens were cast in waterproof plywood forms in an upright position. The column reinforcement was placed in position with the use of steel chairs. The bottom reinforcement for the slab or beam was supported on small wood blocks resting on the bottom of the forms.

All concrete was mixed from 3 to 5 min in a nontilting, horizontal, 6.5-cu ft drum mixer. Slump was determined immediately after mixing. After the concrete was placed in the forms and control cylinder molds, it was vibrated for approximately 15 sec with an internal high frequency rod vibrator.

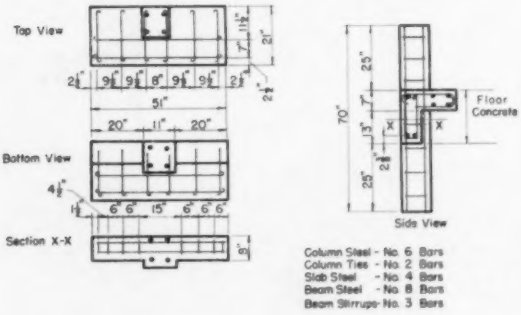
Top View
Bottom View
Section X-X
Side View
Floor Concrete
Column Steel - No. 6 Bars
Column Ties - No. 2 Bars
Slab Steel - No. 4 Bars
Beam Steel - No. 8 Bars
Beam Stirrups - No. 3 Bars

Fig. 6—Details of edge column specimens, Type E (beam series)

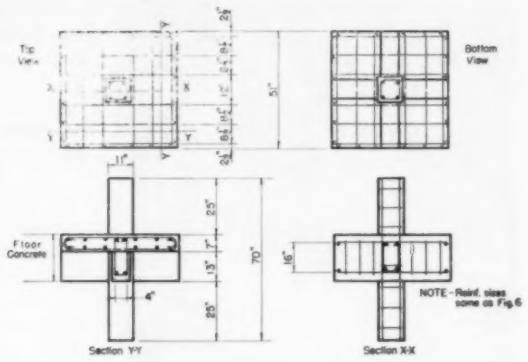

Fig. 7 — Details of interior column specimens, Type I (beam series)

Each specimen in the column series was cast in 1 day. Each specimen required two batches of concrete, and for each batch, six 6 x 12-in. concrete control cylinders were made. The forms were removed the day after casting, and the specimen placed in a fog room at 75 F for 6 days. It was then stored in the air of the laboratory until tested at an age of 28 days.

In the slab series and the beam series, 3 days were required to cast each specimen. The bottom column was cast the first day, the floor portion the second day, and the top column the third day. Each portion of the specimen required one or two batches of concrete, and for each portion of the specimen, six 6 x 12-in. concrete control cylinders were made. On the third day after casting the bottom column, the forms were removed, and the specimen was placed in a fog room at 75 F for 5 days. This, in effect, resulted in 7 days moist curing on the floor portion of the specimen. It was then stored in the air of the laboratory and tested when the floor concrete was 28 days old.

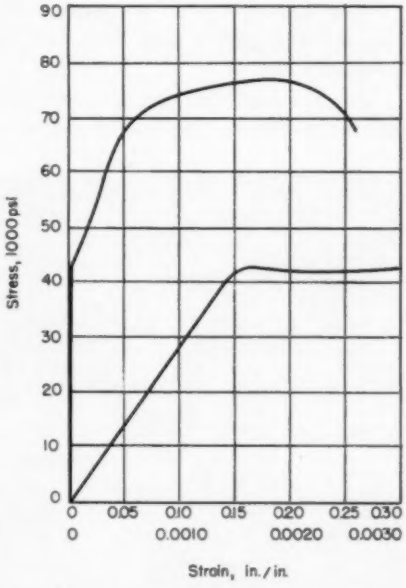

Fig. 8—Typical stress-strain curve for column reinforcement

Test procedure

All specimens were tested in compression in a 3,000,000-lb capacity hydraulic machine. A typical test set-up is shown in Fig. 9. Each specimen was placed centrally under the loading head of the testing machine, and a high strength gypsum cement placed between the bottom column stub and the flat steel bed of the testing machine to insure a smooth bearing surface. The top column stub was capped with a high strength gypsum cement and an 11 x 11 x 1½-in. steel plate. The capping compound was allowed to dry for approximately 1 hr before any load was applied to the specimen.

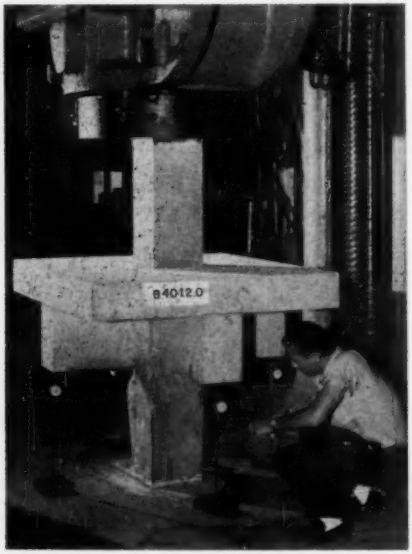

Fig. 9—Test set-up

To insure that the specimens were being loaded axially, three or four dial gages, graduated directly to 0.001 in., were placed around the perimeter of the specimens. The dial gages were mounted on posts which were set on bed of testing machine as shown in Fig. 9.

A preload of 20,000 lb was applied to the specimen before the spherical seat, attached to the head of the testing machine, was wedged in a fixed position to prevent any rotation. The column portions were thus loaded as *flat-ended columns*. The load was then increased to 50,000 lb and in increments of 50,000 lb thereafter until failure occured. At each loading increment, deflection measurements were taken. In addition, the specimen was examined for cracks, and in most cases, a pictorial record was made of changes in the crack pattern. Testing of each specimen took approximately 1½ to 2 hr.

TEST RESULTS

Test data

The test data, for specimens in the column series and for specimens in the slab series and beam series, are summarized in Tables 2 and 3, respectively.

Table 2 includes the concrete strengths of the bottom and top portions of the column, the yield point and ultimate strength of the column

TABLE 2—MEASURED AND COMPUTED QUANTITIES—COLUMN SERIES

Specimen No.	Concrete strength*, f'_{cc}, psi			Tensile strength of column steel, psi		P_{test}, lb	$\frac{P_{test}\dagger}{P_{calc}}$
	Bottom of column	Top of column	Avg.	Yield strength	Ultimate strength		
Column series — Type 57							
C7557	7100	7240	7170	49,410	76,770	748,000	0.92
C6057	5650	5000	5330	44,150	75,190	640,000	1.04
C5057	4690	5180	4940	44,840	77,030	586,000	1.01
C4557	4630	4690	4660	44,980	77,050	560,000	1.02
C3757	2960	3300	3130	48,360	80,380	408,000	1.01
C3057	2800	2710	2760	46,830	76,820	345,000	0.95
Column series — Type 70							
C7570	6450	6820	6640	45,180	76,960	709,000	0.94
C5070	4050	4860	4460	44,750	76,355	548,000	1.03
C3070	2490	2430	2460	45,870	79,300	357,000	1.08
					Average		1.00
					Standard deviation		0.049

*28-day compressive strengths.
†$P_{calc} = A_s f_y + A_c C f'_{cc}$ — Calculated load on the specimen using the 1956 ACI Code ultimate load column formula.

reinforcement, and the ultimate load, P_{test}. The average column concrete strength listed was determined by averaging the bottom column and top column concrete strengths.

Table 3 includes the bottom column and top column concrete strength, floor concrete strength, the ratio of the column concrete strength to the floor concrete strength, the yield point and ultimate strength of the column reinforcement, the location of the failure, and the ultimate load, P_{test}. In computing the ratio of the column concrete strength to the floor concrete strength the compressive strength of either the bottom or top column, which ever was lower, was used.

Behavior of specimens under load

For each type of specimen tested various stages of cracking were observed prior to failure. These are shown in Fig. 10 through 14. In all cases failure occurred by crushing of concrete and yielding of the column reinforcement. The typical behaviors, before and at failure, of the specimens tested were as follows:

Column series — Vertical cracks first appeared in the column faces at loads approaching failure. Final failure occurred by yielding of the column reinforcement and crushing of the concrete.

Slab series — For the sandwich column specimens, Type S, vertical cracks first appeared in the slab concrete between the two columns. Failure occurred in the slab concrete sandwiched between the two column sections.

For the corner column and edge column specimens, Types C and E, three stages of cracking were observed: vertical cracks appearing on the exterior face, or faces, of the slab concrete which were flush with the top and bottom columns; cracking of the slab around the perimeters of the top and bottom columns; and cracks extending from the crack around the perimeter of the column towards the slab edges directly over the slab reinforcement.

Fig. 10—Typical cracking for corner column specimens, Type C (slab series)

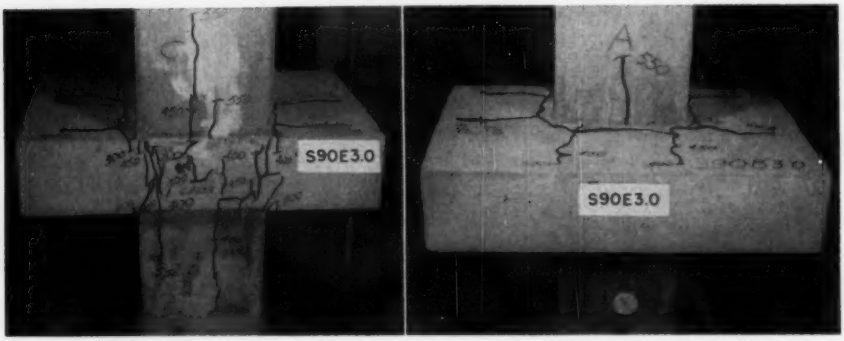

Fig. 11—Typical cracking for edge column specimens, Type E (slab series)

In Specimens S40C2.0, S45C1.5, and S30C1.5, cracks extending from the perimeter of the columns, did not form. Typical cracking patterns are shown in Fig. 10 and 11 for the Type C and Type E specimens, respectively.

Ultimate failure of all Type C and Type E specimens occurred either in the slab portion flush with the column face or faces, or in the bottom column or top column. In Specimens S40E2.0 and S50E2.0, failure occurred at the junction of the bottom column and slab.

For the interior column specimens, Type I, four stages of cracking were observed as shown by a typical example in Fig. 12. Cracking first took place on the vertical faces on the outside perimeter of the over hanging slab indicating that tensile stresses were present on these faces. These tensile stresses were caused by the lateral internal compressive stresses,

TABLE 3—MEASURED AND COMPUTED QUANTITIES— SLAB SERIES AND BEAM SERIES

Specimen No.	Concrete strength,* psi			f'_{cc}†	Tensile strength of column steel, psi		Location‡ of failure	P_{test}, lb	f'_{cp}§, psi	$\frac{P_{test}}{P_{calc}}$**
	f'_{co} Bottom	f'_{co} Top	f'_{cf}	f'_{cf}	Yield strength	Ultimate strength				
Slab series — Type S										
S75S3.0	5330	6090	2180	2.44	48,360	80,380	S	415,000	3250	1.00
S60S2.4	5190	5150	1960	2.63	46,300	78,400	S	376,000	2910	0.99
S50S2.0	5210	5310	2180	2.39	43,330	74,440	S	416,000	3350	1.02
S37S1.5	3180	3020	1960	1.54	45,270	78,570	S	390,000	3060	1.03
								Average		1.01
								Std. deviation		0.011
Slab series — Type C										
S90C3.0	8120	7540	2470	3.05	44,930	77,190	S & BC	480,000	3970	0.90
S75C3.0	7420	7560	2690	2.76	44,700	77,530	S & BC	516,000	4330	0.95
S60C3.0	5510	5380	1280	4.20	46,310	80,640	S	385,000	3010	1.03
S60C2.0	6630	6750	3600	1.84	44,320	76,800	S & BC	550,000	4670	0.96
S50C2.0	5710	5540	2550	2.17	46,080	80,640	S	450,000	3670	0.97
S40C2.0	4590	3510	1510	2.32	45,390	75,110	S	367,000	2840	1.17
S45C1.5	3990	4290	2730	1.46	42,690	76,030	BC	440,000	3600	0.92
S37C1.5	3580	3270	2300	1.42	44,980	78,080	TC	400,000	3170	0.98
S30C1.5	2840	2390	1530	1.57	43,850	75,870	TC	300,000	2210	0.94
								Average		0.98
								Std. deviation		0.076

TABLE 3 (Cont.)—SLAB SERIES AND BEAM SERIES

Specimen No.	Concrete strength,* psi			f'_{ct}	Tensile strength of column steel, psi		Location‡ of failure	P_{test}, lb	f'_{cp}§, psi	P_{test}** / P_{calc}
	f'_{co} Bottom	f'_{co} Top	f'_{cf}	f'_{cf}	Yield strength	Ultimate strength				
Slab series — Type E										
S90E3.0	8060	7610	2440	3.12	44,420	76,050	BS & BC	568,000	4850	0.96
S75E3.0	6800	7700	2380	2.86	45,640	79,360	BS	534,000	4480	0.96
S60E3.0	6340	5190	1720	3.01	46,770	80,410	BS	442,000	3560	1.01
S60E2.0	6600	6540	3470	1.89	44,930	76,500	BS	582,000	4970	0.94
S50E2.0	5120	5380	2350	2.18	45,480	81,200	BS & BC	453,000	3690	0.96
S40E2.0	3970	3360	1390	2.42	44,650	77,910	BS & BC	363,000	2820	1.11
S45E1.5	3450	3670	2560	1.35	43,780	74,650	BC	446,000	3650	1.05
S37E1.5	3420	3020	1990	1.52	44,000	75,350	TC	400,000	3190	1.05
S30E1.5	2400	2290	1470	1.56	45,390	78,110	BC	347,000	2650	1.12
								Average		1.02
								Std. deviation		0.066
Slab series — Type I										
S90I3.0	7400	7400	2480	2.98	46,870	80,740	TC & S	700,000	6110	0.95
S75I3.0	7440	7750	3220	2.31	42,730	74,320	BC	739,000	6550	0.97
S75I3.0	6280	6260	2300	2.72	44,240	76,180	TC	650,000	5630	1.01
S60I3.0	6570	6840	2080	3.17	43,640	75,110	BC	690,000	6050	1.05
S60I2.0	6620	6810	3430	1.93	45,450	78,860	TC	700,000	6140	0.98
S50I2.0	5890	6120	3090	1.91	45,000	77,500	BC	580,000	4940	0.90
S50I2.0	4990	5260	2210	2.26	43,330	74,440	BC	550,000	4660	1.01
S40I2.0	4590	3760	2460	1.53	45,160	76,730	TC	432,000	3490	0.94
S45I1.5	4990	4970	2870	1.73	44,520	77,170	TC	600,000	5150	1.06
S37I1.5	3940	3270	2200	1.49	44,240	76,960	TC	450,000	3680	1.09
S30I1.5	3710	3790	1940	1.91	43,150	76,250	BC	445,000	3640	1.03
								Average		1.00
								Std. deviation		0.034
Beam series — Type E										
B90E3.0	8030	7230	2600	2.78	45,890	77,850	BB	452,000	3670	0.97
B75E3.0	6720	7110	3140	2.14	45,390	78,800	BB	400,000	3170	0.75
B60E3.0	6180	5640	2020	2.79	45,280	77,650	BB & B	400,000	3170	1.06
B60E2.0	5160	5010	2820	1.77	46,140	79,350	BB	488,000	4020	1.01
B50E2.0	5090	4740	2200	2.16	46,410	80,580	BB	392,000	3070	0.98
B40E2.0	3420	3300	1510	2.19	45,150	77,470	BB	344,000	2620	1.16
B45E1.5	4930	4560	2570	1.77	42,655	77,550	BB	451,000	3710	1.02
B37E1.5	4470	4280	2300	1.86	46,030	78,250	BB	428,000	3430	1.05
B30E1.5	2580	1910	1720	1.11	45,760	79,790	TC	296,000	2140	1.08
								Average		1.01
								Std. deviation		0.095
Beam series — Type I										
B60I2.0	4050	4780	2160	1.88	45,500	76,980	BC	542,000	4570	1.11
B50I2.0	4360	3790	2070	1.84	43,340	74,500	TC	496,000	4150	1.08
B40I2.0	3380	3410	1730	1.95	45,020	77,810	BC	450,000	3670	1.07
								Average		1.09
								Std. deviation		0.083
								Over-all average		1.01
								Over-all standard deviation		0.075

*f'_{cc} bottom column—29-day compressive strength; f'_{co} top column—27-day compressive strength; f'_{cf} — 28-day compressive strength.

†Concrete strength, f'_{cc}, top or bottom column, whichever is lower used in ratio.

‡B—beam between slab and column; BB—junction of bottom column and beam; BC—bottom column; BS—junction bottom column and slab; S—slab between column stubs; TC—top column.

§$f'_{cp} = (P_{test} - A_s f_y)/CA_c$. Computed concrete strength in the column at the maximum load where $C = 0.85$.

**$P_{calc} = A_s f_y + A_c C f'_{cp}$. Calculated load on the specimen where $C = 0.85$ and the value f'_{cp} as given by equations on Fig. 15 through 20.

Fig. 12—Typical cracking for interior column specimens, Type I (slab series)

or internal pressure, subjected to the overhanging floor concrete when the floor concrete intersecting the column was loaded axially. The remaining three stages of cracking were as follows: vertical cracks of stage one progressing inward, in line with the slab reinforcement to the perimeters of the top and bottom columns; cracking of the slab around the perimeters of the top and bottom columns; and cracks appearing in the column stubs adjacent to the slab surface.

Failures for Type I specimens occurred either in the top column or the bottom column.

Beam series — For the edge column and interior column specimens, Types E and I, three stages of cracking were observed as shown by typical examples in Fig. 13 and 14, respectively. The first two stages for the Type E and Type I specimens were similar to the first two stages of cracking for the Type E and Type I specimens of the slab series, respectively. Splitting of the beams from the bottom of the slab constituted the third stage of cracking.

Typical failures of the Type E specimens occurred either in the bottom column or in the bottom of the beam next to the column. Specimen B30E1.5 failed in the top column. Typical failures of the Type I specimens occurred in either the top column or bottom column.

ANALYSIS

Ultimate load capacities

Without intersecting floor concrete — The ultimate strength of a short tied column without intersecting floor concrete, subjected to axial loads, is usually considered to be the sum of the ultimate strength, $CA_c f'_{cc}$, of the net concrete area, and the load, $A_s f_y$, required to stress the column

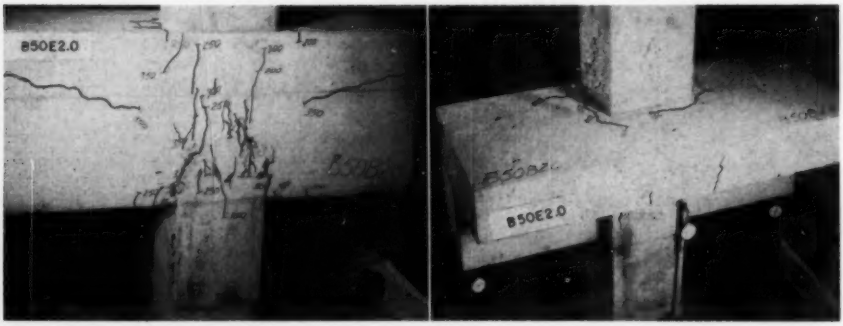

Fig. 13—Typical cracking for edge column specimens, Type E (beam series)

reinforcement to its yield point. The ultimate load, P, which the column can carry may then be computed as follows:

$$P = CA_c f'_{cc} + A_s f_y \quad \text{.....(1)}$$

where C is the ratio of the concrete stress on an axially loaded column to the column concrete compressive strength as determined by 6 x 12-in. cylinders.

Fig. 14—Typical cracking for interior column specimens, Type I (beam series)

The ultimate load capacities for columns without intersecting floor concrete, computed from Eq. (1) with $C = 0.85$,* are compared in Table 2 with the test data. The average ratio of the test to calculated values is 1.00, with a standard deviation of 0.049.

With intersecting floor concrete — In computing the ultimate load of a column, when a sufficient length of the central portion of the column concrete is replaced by the weaker strength floor concrete, the value of f'_{cc} in Eq. (1) should be that of the weaker floor concrete strength, f'_{cf}. However, as the length of the floor section is reduced and restraints provided on the section in the form of slab or slab and beam reinforcement, column reinforcement, surrounding floor concrete, and surrounding column concrete, there will appear to be an apparent increase in strength of the floor concrete above the value expected on the basis of known cylinder strength. This apparent strength, f'_{cp}, may be expressed as follows:

$$f'_{cp} = F(f'_{cf}, f'_{cc}, R_l, R_c, G, p_c, \text{ and } p_f) \quad \text{.....(2)}$$

Other variables may influence the magnitude of f'_{cp} but the ones listed seem to be the important ones. Therefore, in computing the ultimate load of a column, with the usual thickness of intersecting floor concrete, the value of f'_{cc} in Eq. (1) should be that of the apparent strength of the floor concrete, f'_{cp}. Using the value of 0.85 for C and f'_{cp} for f'_{cc} in Eq. (1) the ultimate load of a tied column with intersecting floor concrete may be computed as follows:

$$P = 0.85A_c f'_{cp} + A_s f_y \quad \text{.....(3)}$$

To use Eq. (3), the value of f'_{cp} must be known. Sufficient information is not available to develop a general expression for f'_{cp} involving all the variables listed in Eq. (2). However, expressions for f'_{cp} were

*ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-56)," ACI JOURNAL, V. 27, No. 9, May 1956 (*Proceedings* V. 52), p. 984.

developed for each type of specimen from the limited data obtained in this investigation.

For any *one particular type* of specimen studied in this investigation, the lateral restraint, the end restraint, the geometry of the specimen, the percentage of column reinforcement, and the percentage of slab or slab and beam reinforcement remained constant. Therefore, Eq. (2) indicates that the apparent floor concrete strength, f'_{cp}, is only a function of the floor strength, f'_{cf}, and the column strength, f'_{cc}, or:

$$f'_{cp} = F\,(f'_{cf}, f'_{cc})$$

Empirical equations for f'_{cp} were developed from the test data of the different types of specimens investigated, and are given in Fig. 15 through 20. In these figures, the ratio of the maximum apparent concrete strength to the floor concrete strength, f'_{cp}/f'_{cf}, was plotted against the ratio of the column concrete strength to the floor concrete strength, f'_{cc}/f'_{cf}. The apparent concrete strength, f'_{cp}, in the ratio f'_{cp}/f'_{cf}, was evaluated, for each specimen tested, by using Eq. (3) with P equal to P_{test}. The term f'_{cc} in the ratio f'_{cc}/f'_{cf}, represents the bottom or top column concrete strength, whichever was lower, as determined from 6 x 12-in. cylinders.

The data on each figure were divided into two distinct failure types, column failures and joint failures. For a given type of specimen the type of failure depended on either the column strength or the ratio of the column concrete strength to floor concrete strength.

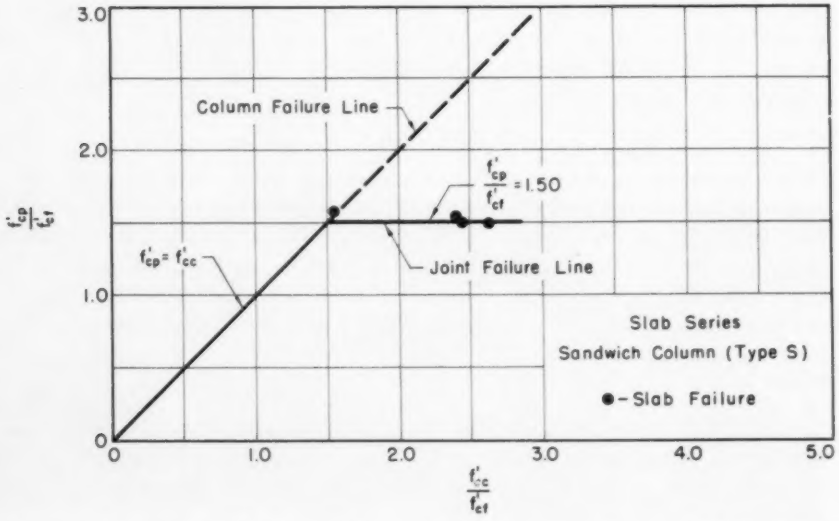

Fig. 15—Effect of f'_{cc}/f'_{cf}, sandwich column specimens, Type S (slab series)

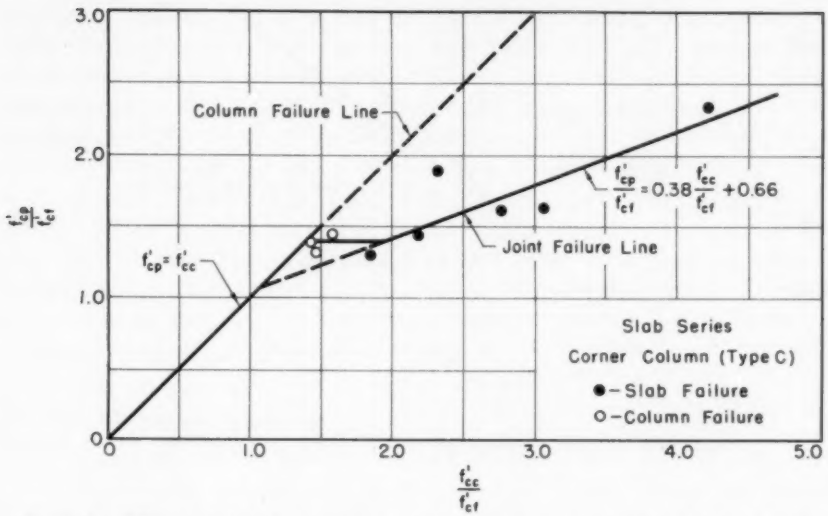

Fig. 16—Effect of f'_{cc}/f'_{cf}, corner column specimens, Type C (slab series)

If the specimen failed in the column, the apparent floor concrete strength, f'_{cp}, in Eq. (3) was equal to the column concrete strength, f'_{cc}. For this case, a 45-deg column failure line was constructed on each figure. In Fig. 15 through 20, it can be seen that all specimens which failed in the column lie close to the column failure line.

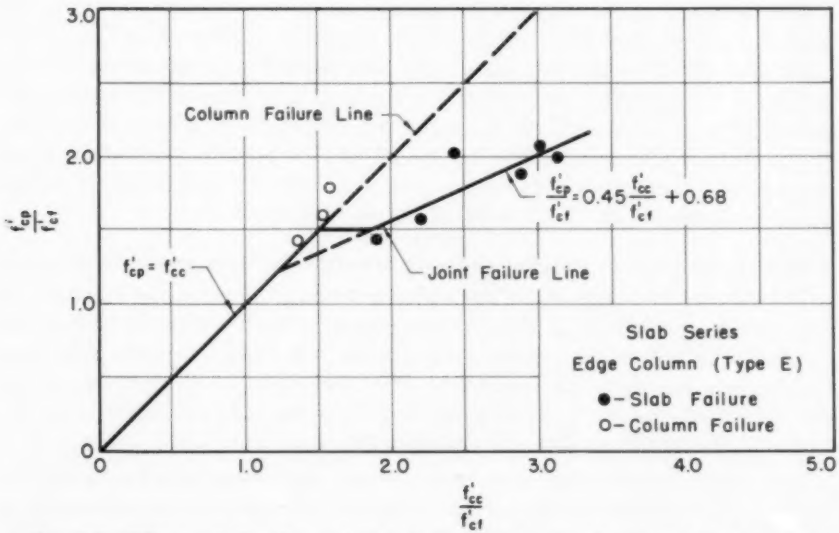

Fig. 17—Effect of f'_{cc}/f'_{cf}, edge column specimens, Type E (slab series)

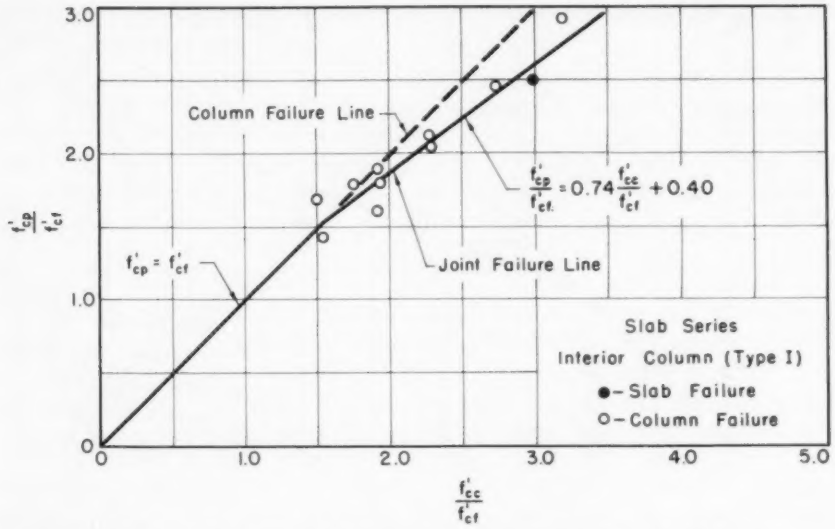

Fig. 18—Effect of f'_{cc}/f'_{cf}, interior column specimens, Type I (slab series)

If the specimen failed in the floor, or joint, the apparent floor concrete strength, f'_{cp}, was less than the column concrete strength f'_{cc}. In this case, a decrease in the load-carrying capacity resulted as compared with a column of the same column concrete strength with no intersecting floor concrete, as indicated in Fig. 15 through 19 by the joint failure lines.

The ratios of the maximum test load, P_{test}, to the calculated maximum load, P_{calc}, are listed for each specimen in Table 3. The value, P_{calc}, was computed by using Eq. (3) where f'_{cp} was given by the equations shown in Fig. 15 through 20. The over-all average ratio of the test to calculated values is 1.01, with an over-all standard deviation of 0.075. The ratios of the test to calculated loads agreed within 12 percent except for three specimens. These specimens were S40C2.0, B75E3.0, and B40E2.0 which had discrepancies of 17, 25, and 16 percents, respectively.

Effect of the ratio of column concrete strength to floor concrete strength

There were two important variables affecting the ultimate strength of the specimens tested in this investigation. These variables were the ratio of the column concrete strength to the floor concrete strength, f'_{cc}/f'_{cf}, and the type of specimen. For the types of specimens tested, the effect of the ratio f'_{cc}/f'_{cf} on the load-carrying capacity may be studied directly from Fig. 15 through 20.

The analysis indicates that up to some maximum critical value of f'_{cc}/f'_{cf}, there will be no reduction in column strength due to intersecting floor concrete. Up to this critical value of f'_{cc}/f'_{cf}, the apparent floor

concrete strength, f'_{cp}, is equal to the column concrete strength, f'_{cc}, thus resulting in a column failure. For values of f'_{cc}/f'_{cf} greater than the critical value, there will be a reduction in the column load-carrying capacity as compared with a column of the same column concrete strength with no intersecting floor concrete. For this case, the apparent concrete strength f'_{cp} is less than f'_{cc}, thus resulting in a joint failure.

The analysis indicates that ratios of f'_{cc}/f'_{cf} of 1.50, 1.40, 1.50, and 1.54 for the Types S, C, E, and I specimens, respectively, of the slab series, and 1.39 for the Type E specimens of the beam series can be reached before the apparent concrete strength, f'_{cp}, becomes less than the column concrete strength, f'_{cc}. For the Type I specimens in the beam series the apparent concrete strength, f'_{cp}, is still equal to the column concrete strength, f'_{cc}, for a ratio of f'_{cc}/f'_{cf} of approximately 2.0

Effect of type of specimen

The effect that the type of specimen had on the load-carrying capacity of a column can be observed in Fig. 21, which consists of the curves shown in Fig. 15 through 20.

For ratios of f'_{cc}/f'_{cf} defining joint failures, the joint failure line for each type of specimen in the slab series and beam series, was at a different slope as shown in Fig. 21. A study of the figure indicates that the slope of the joint failure line increased as the number of sides of the specimen being restrained by overhanging floor concrete increased. The slopes of the joint failure lines were 0.00, 0.38, 0.45, and 0.74 for the

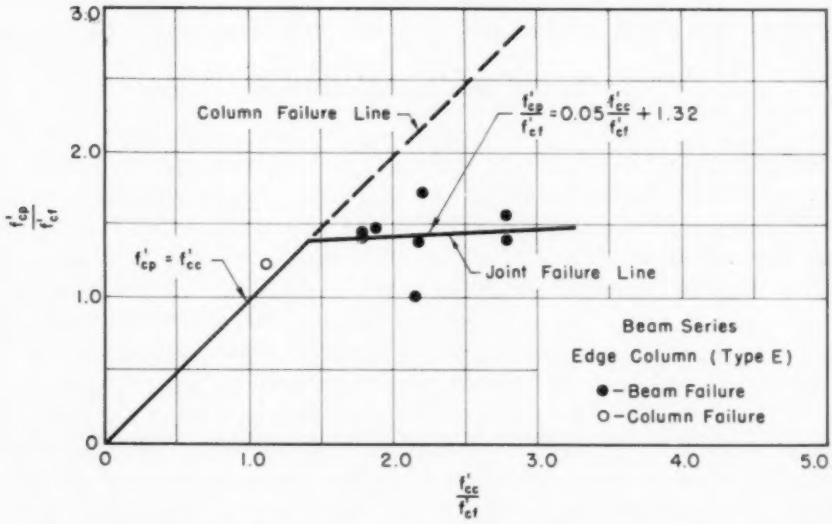

Fig. 19—Effect of f'_{cc}/f'_{cf}, edge column specimens, Type E (beam series)

Types S, C, E, and I specimens, respectively, of the slab series and 0.05, and 1.0 for the Types E and I specimens, respectively, of the beam series. It is interesting to note that Types C and E specimens of the slab series had a joint failure line slope of 0.00 for ratios of f'_{cc}/f'_{cf} less than 1.95 and 1.82, respectively.

In the Type S specimens of the slab series, the apparent concrete strength reached a uniform top limit of 1.50 f'_{cf} for ratios of f'_{cc}/f'_{cf} greater than 1.50. It is believed that the magnitude of the top limit will depend on the dimensions of the weaker concrete, and to some extent on the actual strength of the weaker concrete.

The data indicate that the efficiency of the Type E specimens of the slab series was greater than the Type E specimens of the beam series. The increase in efficiency may be explained in terms of the thickness of weaker concrete and the amount of lateral restraint on the weaker floor concrete. In the Type E specimens of the slab series the weaker concrete was restrained for its full depth on three sides, whereas, in the Type E specimens of the beam series the weaker concrete extended down the column shaft below the slab for the depth of the beam and was restrained on only two sides and a small portion of the third side.

The efficiency of the Type I specimens of the beam series was slightly greater than the Type I specimens of the slab series. However, broad conclusions should not be drawn because statistically there was an insufficient amount of test data for the Type I specimens of the beam

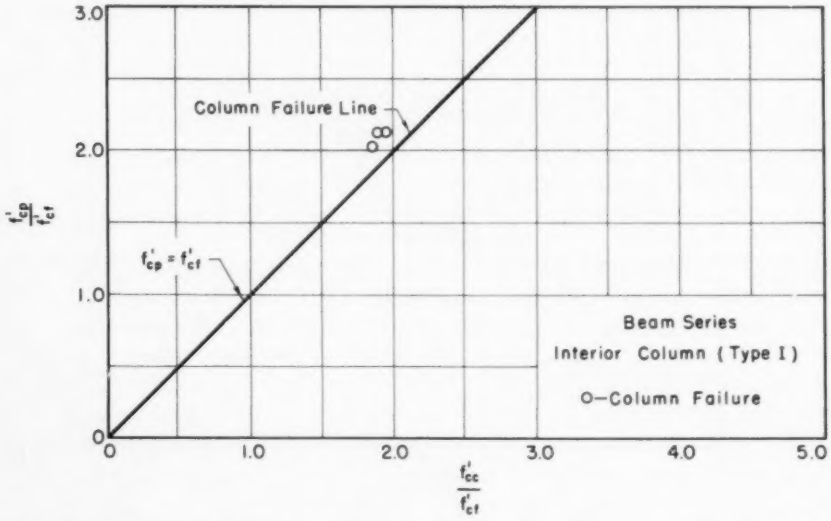

Fig. 20—Effect of f'_{cc}/f'_{cf}, interior column specimens, Type I (beam series)

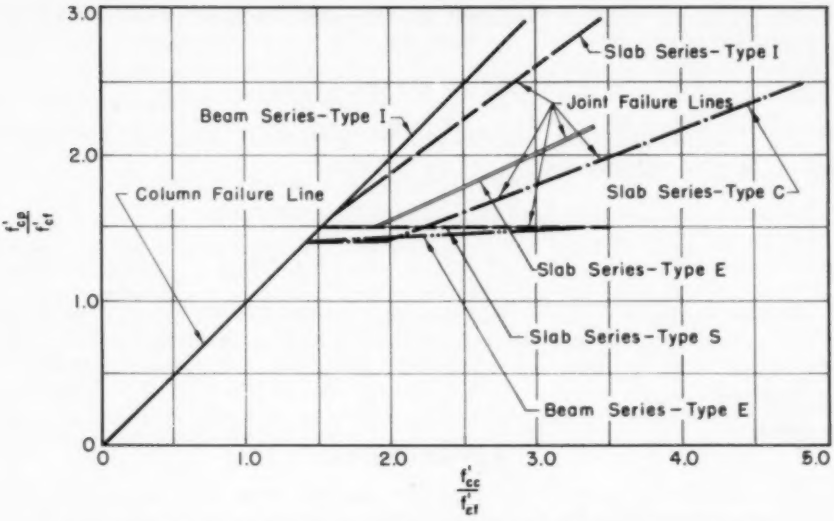

Fig. 21—Effect of type of specimen (slab series and beam series)

series to evaluate a joint failure line. An examination of Fig. 20 indicates that only three Type I specimens of the beam series, with approximately the same ratio of f'_{cc}/f'_{cf}, were tested. However, based on the data obtained, it may be possible that the restraint to the weaker concrete strength was greater in the Type I specimens of the beam series due to the extra projection of concrete and use of additional steel as compared to the restraint in the Type I specimens of the slab series.

DESIGN CONSIDERATIONS

Design formulas

Based on the results of this limited investigation using sand and gravel concrete, *and assuming tentatively that the results are applicable to all tied columns,* the present ACI Code tied column formulas, both for working load design and ultimate strength design may be used for ratios of f'_{cc}/f'_{cf} up to 1.4 for corner and edge columns and for ratios of f'_{cc}/f'_{cf} up to 1.5 for interior columns.

For corner and edge columns, the data indicate that no substantial benefits may be obtained by increasing the column concrete strength beyond 1.4 times the floor concrete strength.

For interior columns it appears that approximately 75 percent of the column concrete strength above 1.5 times the floor concrete strength may be effective in sustaining load. For this case the value of f'_c to be used in both the working load and ultimate strength tied column

formulas listed in the ACI Code* may be computed from the following equation:

$$f'_c = 1.50\,f'_{cf} + 0.75\,(f'_{cc} - 1.50\,f'_{cf}) \quad (4)$$

Using f'_c as given by Eq. (4) the following working design and ultimate strength tied column formulas are obtained:

$$P_a = A_g\,(0.14\,f'_{cc} + 0.067\,f'_{cf} + 0.80\,f_s p_g) \quad (5)$$

$$P = A_c\,(0.64\,f'_{cc} + 0.31\,f'_{cf}) + A_s f_y \quad (6)$$

The data indicate that Eq. (5) and (6) are applicable for ratios of f'_{cc}/f'_{cf} between 1.5 and 3.0 for interior slab intersected columns and for ratios of f'_{cc}/f'_{cf} between 1.5 and 2.0 for interior slab and beam intersected columns.

CONCLUSIONS

The following conclusions may be drawn from the results of this limited investigation on the effect of floor concrete on column strength.

1. Two types of failures were observed in these tests: column failures and joint or floor failures. The type of failure obtained in each specimen depended on either the column strength or on the ratio of the column concrete strength to the floor concrete strength.

2. The data and analysis indicate, that up to some critical value of the ratio of column concrete strength to floor concrete strength f'_{cc}/f'_{cf}, the 1956 ACI Building Code ultimate load column formula may be used in computing the ultimate load for tied columns with intersecting floor concrete. These critical ratios of f'_{cc}/f'_{cf} are 1.4 for corner and edge columns and 1.5 for interior columns.

3. For ratios of column concrete strength to floor concrete strength greater than the critical values, the ultimate strength of a tied column with intersecting floor concrete is a function of the amount of restraint offered to the floor portion of the column and also of the ratio of the column concrete strength to the floor concrete strength. For this case the ultimate strength may be computed from Eq. (3) which is a modified form of the present ultimate load column formula.

The above conclusions are based on a limited investigation in which only two variables were considered. These were the ratio of the column concrete strength to the floor concrete strength and the type of joint. Many items which might have considerable influence on the behavior of the specimens, such as the type of column (tied or spiral), column size and shape, percentage of column reinforcement, thickness of the floor, percentage of floor reinforcement, the height of the specimen,

*ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-56)," ACI JOURNAL, V. 27, No. 9, May 1956 (*Proceedings* V. 52), p. 984.

type of concrete (lightweight or normal weight), and the type of loading (axial or eccentric), were not considered. A broader investigation should be conducted in which the effects of the above variables, constant in this investigation, should be studied to provide a better understanding of the behavior involved and a more adequate basis for design.

ACKNOWLEDGMENT

This investigation was carried out as a project of the Engineering Experiment Station in the Department of Theoretical and Applied Mechanics at the University of Illinois, in cooperation with the Reinforced Concrete Research Council of the Engineering Foundation. Credit for initiating the investigation must be given to the late J. DiStasio, Sr., who planned the tests in cooperation with Dr. I. M. Viest, formerly research associate professor in the Department of Theoretical and Applied Mechanics, University of Illinois. A task committee consisting of J. DiStasio, Sr., chairman; R. C. Reese; and C. A. Willson, was appointed by the Council to provide supervision of the investigation. After the untimely death of Mr. DiStasio, Mr. Reese became chairman of the task committee.

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Title No. 56-59

Performance and Design of Special Purpose Blast Resistant Structures

By ROBERT A. WILLIAMSON

The structures housing scientific equipment used in nuclear tests in the South Pacific are discussed from the standpoint of service experience and structural design approach. Two structures, one sited below ground, and the other above ground, are described, and damage history is briefly discussed. The general procedure currently used in the design of these structures is presented and illustrated with a numerical example.

PROBABLY THE LARGEST AMOUNT of concrete construction of American origin, designed to resist the effects of nuclear blast, is located beyond the limits of the United States, on the coral islands of the Eniwetok and Bikini Atolls, in the South Pacific. Most of this construction consists of scientific stations, housing measuring and support equipment used to obtain information for scientists in studies of nuclear fission and fusion phenomena

Design of these structures often involves problems stemming from extremely high overpressure and extrapolation of available knowledge, while construction on the remote and scattered island complex presents additional problems in material supply and logistics. Both design and construction were performed under a tight timetable imposed by conditions typical of nuclear test operations.

The service record of these stations constitutes an almost unrecognized, but sizable body of information. This paper deals with the performance of some of the structures and the approach currently used in their design.

STRUCTURAL TYPES

The stations can be broadly classified according to strength, into non-expendable or semi-expendable types, depending on whether survival in an undamaged condition is required, or whether damage is acceptable provided that records and equipment can be recovered intact. However, the distinction between these two categories is not clearly defined.

Robert A. Williamson, ACI member, is chief structural engineer for Holmes and Narver whom he joined in 1950. He designed and evaluated many of the blast resistant structures used in the nuclear bomb tests at Eniwetok and Nevada proving grounds which were conducted by the Atomic Energy Commission. Prior to this he spent eight years as a structural designer of commercial, industrial, and military projects and five years in the field of aircraft design.

Nearly all the structures are of reinforced concrete, made using coral aggregate and sea water.[1] This material is stable in the climatic conditions which occur, and provides a good balance between strength and protection from radiation. Ultimate static strengths of 3000 psi are normally used in design. Concrete utilizing limonite ore for fine aggregate and steel scrap for coarse aggregate, and mixed using sea water, were used where dictated by shielding requirements.[2] A typical concrete of this type has a density of about 300 lb per cu ft and a strength of 2500 psi.

The structural systems used were influenced largely by the requirements of the scientific agency involved, by construction conditions, and by the "crash" nature of the test operation. Circumstances generally result in the choice of one and two way slabs for walls, roof, and foundations; however, flat slab roof systems (usually without drop panels) are occasionally used. The high water table in many cases dictates the choice of a mat footing.

The size and configuration of these stations varies widely, so that it is difficult to select any structure as being representative. However, two major stations, one buried, the other located on the surface, have been chosen as examples. Both of these stations, like most of the others, were unmanned at the time of the shot, all equipment operations being remotely controlled by a sequence timing control system.

Fig. 1 shows the general configuration of an earth covered station, consisting of three instrumentation rooms, A, B, and C, and utility room,

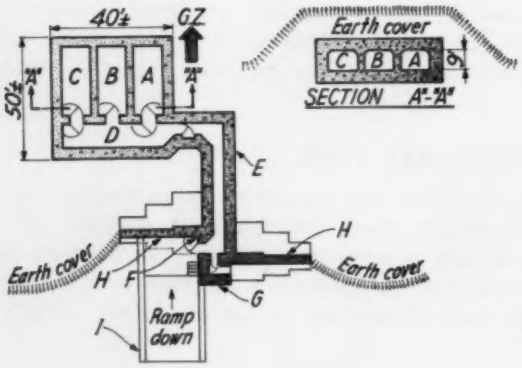


Fig. 1—Floor plan of earth covered recorder station

D, which houses support equipment, including motor generator sets and dehumidifying units. These rooms occupy an area about 40 x 50 ft and have a 9 ft ceiling height. (The arrow marked GZ near the top of the figure points toward Ground Zero.) A tunnel, E, with a 90 deg bend to reduce radiation effects, provides normal access to the station through blast door F. Emergency access is available through the vestibule, G, which provides postshot radiation protection for personnel engaged in recovery of records.

Because of the high water table, the station is only semi-buried, the earth cover over the roof being supplied by backfill confined by retaining walls, H, which are expendable under blast. The area I is a ramp and loading platform, also expendable.

Lead anti-radiation doors are provided at entrances to rooms A, B, C, and D. A section through rooms A, B, and C is shown in the upper right part of this figure. Throughout this area roof and floor thicknesses are 48 in. and exterior wall thicknesses are 42 in.

Roof, floor, and walls of the tunnel are 34 in. thick. Steel percentages throughout the structure range from about 0.25 to about 0.6. All elements are designed as one and two way slabs.

Fig. 2 shows the general features of a multistory camera bunker located on the ground surface. Plan dimensions are roughly 56 x 33 ft;

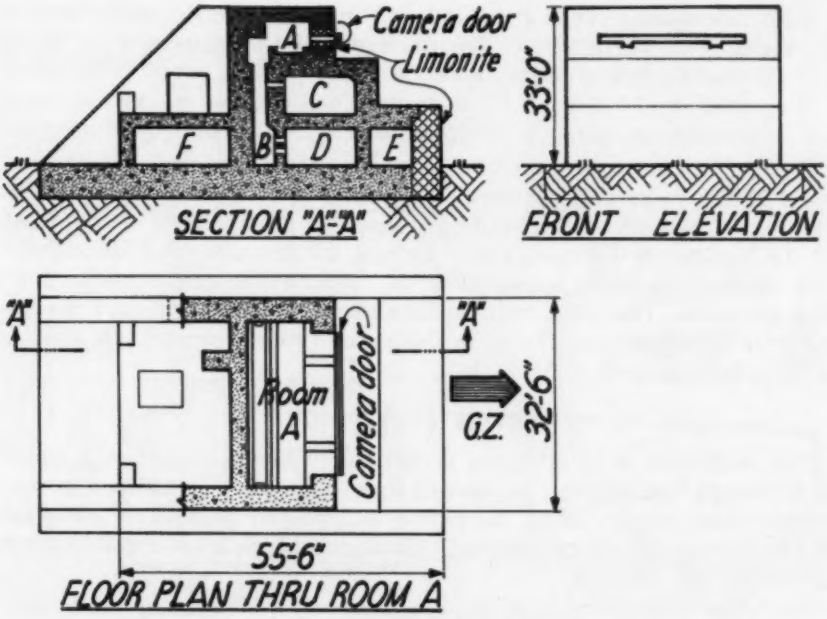


Fig. 2—Camera bunker

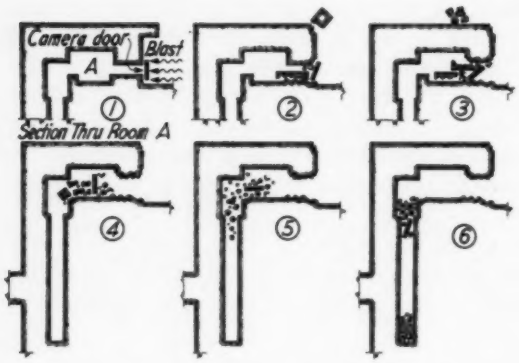


Fig. 3—Failure of camera door in camera bunker

height above grade is also 33 ft. The camera door opening, which faces Ground Zero, is a narrow horizontal slot 18 in. high and 24 ft long, closed by a steel door which is hinged at the top and held in the open position by a cable incorporating an explosive link. This link is detonated on receipt of a timing signal, breaking the connection, and allowing the door to swing shut prior to the arrival of the blast wave. Room A and the vertical shaft B contain reflecting mirrors; C, D, and E are rooms housing high-speed cameras and associated equipment, and F is a utility room.

Roof thicknesses range from 30 to 72 in., and exterior walls vary in thickness from 36 to 72 in. The thickness of the buttresses is 36 in. and the mat footing is 58 in. thick. Steel percentages range from about 0.25 in thick walls to about 0.7 in thinner walls. Cross hatched areas in the figure indicate limonite concrete, used for radiation shielding.

Except for the lintel beam over the camera door and beams over room F, the construction consists generally of one and two-way slabs. Floors and roof act as diaphragms, transmitting blast loads on the front of the bunker to the sidewalls. Ties and stirrups are used throughout and diagonal bars are added in major construction joints where high stresses occur. The joint between the limonite front wall and the remainder of the structure requires this type of reinforcement and the use of shear keys as well.

SERVICE EXPERIENCE

The performance of all types of structures has been generally satisfactory at pressures often far beyond those commonly considered in protective construction. With very few exceptions, parital or complete failure has usually been confined to features which were not important functionally.

The most common type of damage occurs to the retaining walls used to confine earth berms covering buried stations. It is usually impractical

to design these walls to remain completely intact; instead, when such retaining walls are loaded by overpressure against the outside face, the backfill is used to limit the wall movement. As would be expected, typical distress consists of cracking at points of restraint, such as those present at junctions with the main structure and at wall returns. However, such damage can be minimized by proper backfilling techniques.

Blast waves approaching from the fill side seem to present no problem. Walls designed for static earth pressure only have remained intact after being exposed to this type of loading in the 100 to 200 psi zone. However, the top part of the wall has collapsed on several occasions when the backfill was terminated below the top, or when the fill had been previously eroded by blast generated water waves.

One major failure has been experienced as shown in Fig. 3. This was the shear failure of the camera door sill in the large surface mounted bunker previously shown, which was exposed to pressure considerably greater than twice the anticipated value. The postulated sequence of events is shown in views 1 through 6, the door finally being wedged in the vertical shaft at the rear of the structure as shown in view 6. The shear stress on a section through the door sill (section A-A of Fig. 4) may have exceeded 600 psi. Minimum concrete cylinder strength in this structure was 3200 psi. Additional damage also occurred in other locations.

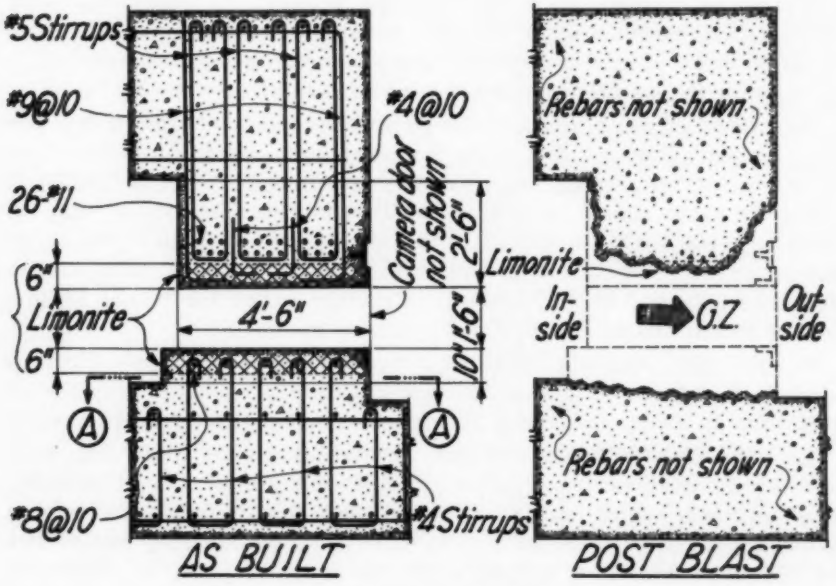


Fig. 4—Section thru camera bunker door frame

Flexural cracks appeared in the outer face of the lintel beam above the camera door at the underside of the roof slab. In addition, shear cracks occurred in the sidewall adjacent to the shutter opening, and in one of the buttresses.

Ground shock due to nuclear explosions has produced damage similar to earthquake damage, in an earth-covered structure located in a 150-200 psi range. This consisted of severe cracking at junctions between distinct parts of the structure, which might have been minimized had it been possible to structurally separate the two parts. This was not feasible, largely because of radiation problems. Horizontal accelerations were sufficient to shear latches on interior lead doors, and bolts anchoring a compressor to its foundation.

DESIGN METHODS

Criteria for designing blast resistant structures is developing so rapidly that periodic modifications of procedures have become necessary. The original approach in designing the scientific structures was a necessary expedient which treated overpressures as static loads and utilized the conventionl working stress concept, using the customery increases in allowable stresses appropriate to short term loading. The service record of stations designed in this way was generally satisfactory; however, the inconsistent amount of reserve strength and lack of economy in this procedure demonstrated the necessity for evaluating ultimate resistance and transient response in a more realistic fashion. To satisfy this need, the original working stress concept was replaced with one based on ultimate strength, and the steady state load assumption was modified to account for the transient nature of the blast loading.

The selection of a revised design method was influenced by two main factors. They were: (1) the accuracy with which the loading and its effect can be predicted and (2) the impelling need for a procedure which is rapid. Load prediction involves considerable error, and, in addition, even slight variations in the loading and structural properties can cause major variations in response. For these reasons, uncertainties as to the behavior of the "as built" structure under the actual loading cannot be eliminated by analytical refinements.

Furthermore, the nature of test operations at Eniwetok Proving Ground requires the design and construction of a large number of structures in a very short time. Structural design methods must be rapid.

The necessity for speed, and lack of precision in the input data both stressed the need for a procedure which trades precision for simplicity. The approach which most nearly fulfilled this requirement is that based on the work of Newmark.[3,4] While this method may not suffice for all structures, it is adequate for the large majority of the types considered here. Its principal features involve (1) the use of a simplified

type of resistance-deflection curve in the elastic and plastic ranges, (2) substitution of a load pulse decaying linearly with time in place of the actual load-time relation, and (3) provision for a rapid means of determining dynamic response. The aim of this design method is to approximate the resistance required in order to survive the transient overpressure with a degree of damage falling within some specified range. The technique consists of (1) estimating the natural period of vibration of the member relative to the effective duration of the blast load, (2) estimating an acceptable limiting value of the maximum transient deflection in terms of the yield point deflection, and (3) from these estimates determining the yield resistance, or equivalent static load for which the design must be made. This approach has been presented before,[3] and the following discussion merely summarizes its main features, and the considerations on which it is based.

RESPONSE PARAMETERS

Notation

A = ratio of energy absorption at maximum deflection to energy absorption at yield

A_s = area of tension steel, sq in.

A_s' = area of compression steel, sq in.

b = beam width, in.

C_t = coefficient in formula for period of vibration, in. per sec

C_y = coefficient in formula for yield deflection

D = dimensionless impulse parameter = $(2\pi I/rT)^2$

d = effective depth, in.

f_c' = ultimate compressive strength of concrete, psi

I = impulse (area under pressure-time curve), psi-sec

L = clear span, in.

M = bending moment, in.-lb

p = overpressure intensity, psi

r = yield resistance of a structural element, psi

T = period of vibration, sec

t = time, sec

t_1 = effective duration of linearly decaying overpressure, sec

y = deflection, in.

μ = ductility factor—ratio of deflection at collapse to deflection at yield

ϕ = percentage of tension steel for positive moment = $100A_s/bd$

ϕ' = percentage of compression steel = $100A_s'/bd$

Subscripts

c = compressive

m = maximum

r = rise time

t = period of vibration

y = yield

First to be considered are the various properties which affect structural response. As shown in Fig. 5, there are several main parameters establishing the dynamic behavior of a structural element, such as the beam indicated, which carries a transient load decaying linearily with time from an initial peak value. One set of parameters consists of the properties of this triangular loading (peak overpressure, p_m, and its duration, t_1); the other set is associated with the structural properties

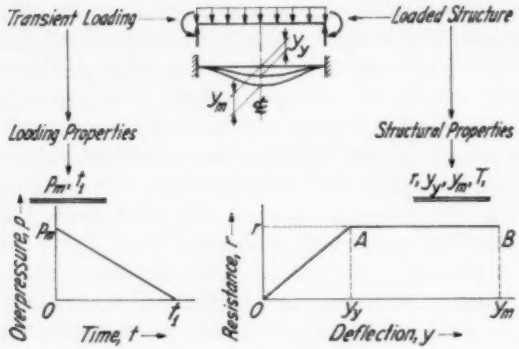


Fig. 5—Response parameters

of the member, in particular, its idealized resistance deflection curve, defined by the peak resistance, r, which the member develops when a static load is applied in the same pattern as the dynamic load, p. This resistance establishes the ordinates of points A and B. The abscissas of these points are the yield deflection, y_y, and the maximum transient deflection, y_m. The ratio, y_m/y_y, cannot exceed a certain upper limit, as governed by the ductility available in the member. At this upper limit, the deflection, y_m, becomes the collapse deflection, and the ratio y_m/y_y, attains its maximum permissible value, μ, which is a structural property of the member called the ductility factor. The final parameter in this set is the fundamental period of vibration of the member, designated as T, which, in part, is a function of the slope of line OA.

The required resistance of a beam, such as the one shown in Fig. 5, is readily found in terms of the other parameters by use of the response chart shown in Fig. 6. This chart, developed by Newmark,[3] gives the response of structures which can be closely represented by undamped, single-degree-of-freedom systems subjected to a triangular load pulse. The plot, based on results of exact analyses, is in dimensionless form, with ordinates in terms of y_m/y_y, and abscissas in terms of the ratio of the effective duration of the loading, t_1, to the period of vibration, T. The solid lines sloping upward to the right are ratios of peak overpressure, p_m, to the resistance, r, and the dashed lines sloping upward to the left are ratios between the time, t_m, to reach maximum deflection, and the period of vibration. Given any two of these parameters, the third may be obtained. For example, if the ratios p_m/r and t_1/T are given, y_m/y_y can be found. Or. conversely, if t_1/T is given, and if y_m/y_y is specified to have any value between unity and its upper limit, μ, the ratio p_m/r can be determined.

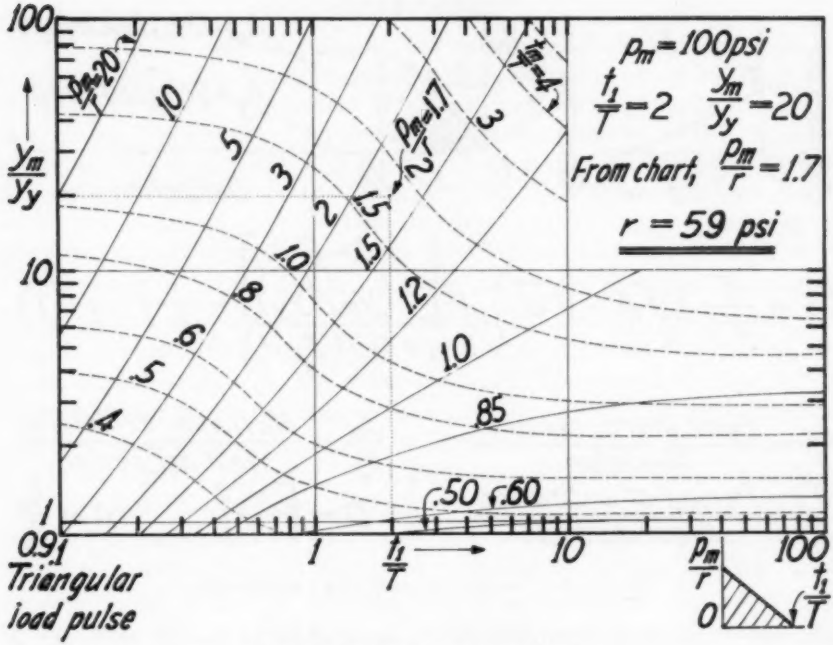
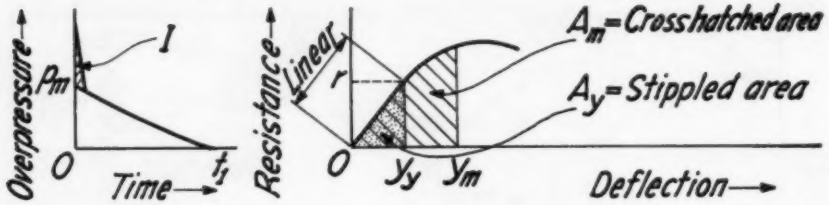


Fig. 6—Dynamic response chart

Since blast loading prediction is subject to variation within wide limits, and since estimates of structural properties also lack precision, it becomes important to determine how changes in the various parameters affect structural response. The data in the chart provides insight of this nature in a form convenient for design purposes. A simple example illustrating use of the chart is shown in the upper right portion of Fig. 6, where it is indicated that if a peak blast pressure of 100 psi intensity has an effective duration equal to twice the period of vibration of a member, and if the maximum transient deflection is to be 20 times the yield deflection, the required resistance is about $100 \div 1.7$ or 59 psi.

Another, more general solution, is used for finding response to a loading of the type shown in Fig. 7, decaying linearly, as before, from its peak value, p_m, to zero, in a time t_1. Superimposed on this curve is an initial impulse — a high intensity pressure lasting for a time which is short compared to the period of vibration, and whose effect is measured by the cross-hatched area, I, under the pressure-time curve. In some cases, this loading condition simulates the actual blast loading more closely than does the triangular variation without the impulse spike. The



$$\frac{p_m}{r} = \frac{T}{\pi t_1}\left(\sqrt{A}-\sqrt{D}\right) + \frac{\left[\frac{A-D}{2\left(y_m/y_y\right)}\right]}{1+0.7\left(T/t_1\right)} \qquad \text{---------(1)}$$

$$\frac{p_m}{r} = \frac{\text{Peak overpressure}}{\text{Resistance}} \qquad \frac{y_m}{y_y} = \frac{\text{Maximum deflection}}{\text{Yield deflection}}$$

$$\frac{T}{t_1} = \frac{\text{Period of vibration}}{\text{Blast duration}} \qquad D = \left(\frac{2\pi I}{rT}\right)^2 \qquad A = \frac{A_m}{A_y}$$

Fig. 7—Semi-empirical response relationship

solution,[5,13] based on straightforward application of energy concepts, is given by Eq. (1), Fig. 7, with terms as defined in the figure.

When t_1 is very small compared with T, the second term vanishes and the first term gives an exact result. At the other extreme, when t_1 is very large compared with T, the first term vanishes and an exact solution is given by the numerator of the second term. The denominator of the second term is an empirically determined adjustment which enables Eq. (1) to be applied throughout the entire range of t_1/T with an error which is usually small.

The D term is associated with the impulse spike, and the A term, which involves areas under the load-deflection diagram, is the ratio of the total energy absorption at failure to the energy stored at yield. When used for cases to which the response chart of Fig. 6 applies, Eq. (1) gives results which are essentially the same, the maximum error being less than 5 percent.

Use of the response chart, or Eq. (1), eliminates much of the work which would otherwise be involved in the rigorous solution of the dynamic problem, so that the major portion of the design effort lies in establishing the response parameters themselves—the loading properties and the structural properties of the member. Much of this effort would still be required even if the actual loads to be considered were applied statically rather than dynamically.

BLAST LOADS

The matter of blast loading is the most uncertain feature of blast resistant design. Unclassified data on this subject is given in the book entitled *The Effects of Nuclear Weapons*,[5] prepared by the Department of Defense, and published by the AEC, and has been extended, in some areas, in two recent ASCE papers by Anderson[6] and Murphy[7] in the November, 1958, issue of the Structural Division Journal. These documents are referred to here because the limited scope of this paper precludes an adequate discussion of blast phenomena.

The variation of pressure in the blast wave with time can take many forms, and is affected by such factors as the yield of the device, the height of burst, horizontal range, and interaction with the ground or water surface, and with the structure itself. The so-called ideal wave form shown in Fig. 8(a), which rises to an instantaneous peak, is only one case. Other, non-ideal forms,[6] such as that shown in Fig. 8(b), may result in a less severe loading on buried structures, but, due to unusually high dynamic pressure, can cause a more severe loading on surface mounted structures of certain types. One common characteristic of the non-ideal wave is its slow rise time, t_r. Small changes in the rise time relative to the period of vibration can bring significant changes in response, making the calculation of maximum deflection extremely unreliable. An example of this is shown in Fig. 9, adapted from analytical studies by J. W. Melin[8] of the University of Illinois. This figure plots maximum transient deflection, y_m, relative to the yield deflection, y_y, for rise times, t_r, which are various multiples of the period of vibration, T. In the particular case illustrated, the triangular pulse has a total duration, t_1, equal to ten times the period of vibration, and the beam is capable of resisting a static uniform load, of intensity, r, equal to that of the peak overpressure, p_m. Under these conditions, which are not extreme, the maximum transient deflection can vary as much as 700

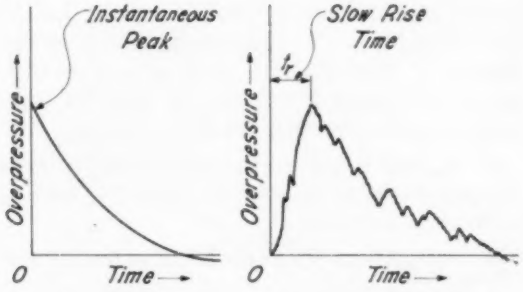


Fig. 8—Blast wave forms

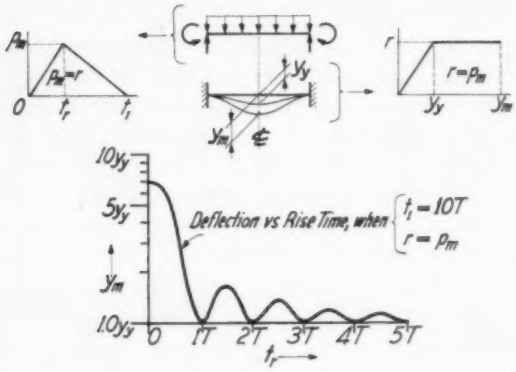


Fig. 9—Effect of rise time

percent, depending on whether the rise time is zero or some integral multiple of the period of vibration.

Since the occurrence of a steep-fronted shock wave with very short rise time may be a possibility, current design practice is to assume zero rise time. It should be noted that finite rise time is only one of several factors which make deflection predictions inaccurate except under closely controlled conditions.

As previously mentioned, a pressure-time relation decaying linearly with time, is substituted for the actual curved variation, for design purposes. The principle involved in the substitution is shown in Fig. 10 and is based on the simple fact that normally only that portion of the loading prior to the instant, t_m, at which maximum deflection occurs, is of interest. Usually, this interval is a very small fraction of the total duration of a nuclear blast, so that the difference between the equivalent linear approximation and the actual curve is also small. Hence, the nearest linear substitution having a duration, t_1, which is compatible with time, t_m, gives acceptable results.

The procedure in making this substitution is as follows:

1. As a first trial, assume t_1 so that the area under the assumed linear approximation roughly equals that under the actual curve (line fa). Calculate t_1/T, and, for the assumed value of either y_m/y_y, or p_m/r, find t_m/T from a chart, such as Fig. 6. Calculate t_m and locate this as point b. If point b falls to the right of line fa, the assumed t_1 value is correct and no further trails are needed.

2. If point b falls to the left of line fa, draw fbc, obtaining a new trial t_1 corresponding to point c. With this new t_1 value find a new t_m value as in Step 1 (point d).

3. Draw fde to get another value for t_1 (*point e*).

4. Repeat Steps 2 and 3 until the trial and derived values of t_1 reasonably agree. The process usually converges very quickly.

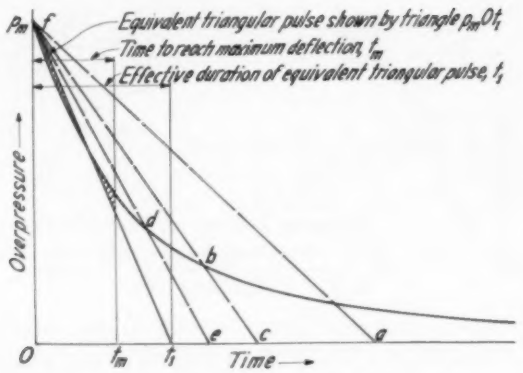


Fig. 10—Determination of equivalent triangular pulse

5. When agreement is reached, as a further refinement, the t_1 value can be adjusted so that the area under the linear approximation up to time t_m equals that under the actual curve, as indicated by the cross hatching in the figure.

Another loading case of practical interest is that shown in Fig. 11, typical of the pressure-time variation on a vertical wall exposed to reflected pressure. If the structure attains its maximum deflection before the overpressure drops to the value p_m, the effective duration can be approximated by point t_1', located by extending line FA. If the time to reach maximum deflection is longer, the load-time curve can be approximated by FAt_1. The value of t_1 can be conservatively estimated by applying the procedure previously demonstrated in Fig. 10 to the curve $p_m AB$, neglecting the impulse spike $Fp_m A$. This results in a loading curve to which Eq. (1) is applicable (see Fig. 7).

An important simplification occurs when the effective duration of the overpressure is many times the period of vibration of the member to be

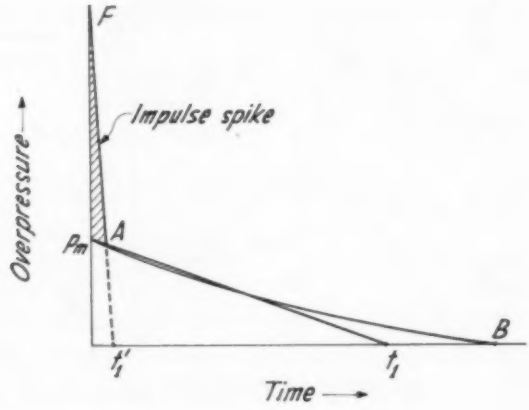


Fig. 11—Equivalent triangular pulse with impulse spike

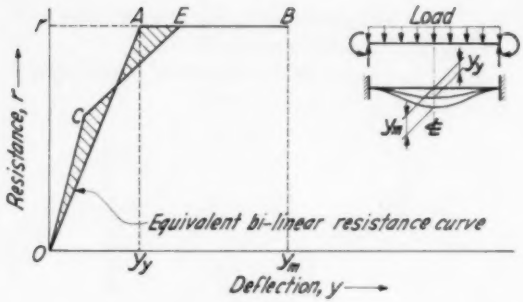


Fig. 12 — Determination of equivalent bi-linear resistance curve

designed. Under these conditions only a rough estimate of the duration is needed, and the member can be designed for a static load equal to the blast load multiplied by an appropriate safety factor. This is the situation which, for example, prevails in the case of buried structures subjected to long duration overpressures typical of detonations in the megaton range.

STRUCTURAL PROPERTIES

Consideration will now be given to the various parameters associated with load-deflection characteristics of the structural element itself. Regardless of whether the member is a beam, two way slab, or any other member which can be represented adequately by a single-degree-of-freedom system, most of the properties defining its behavior are known

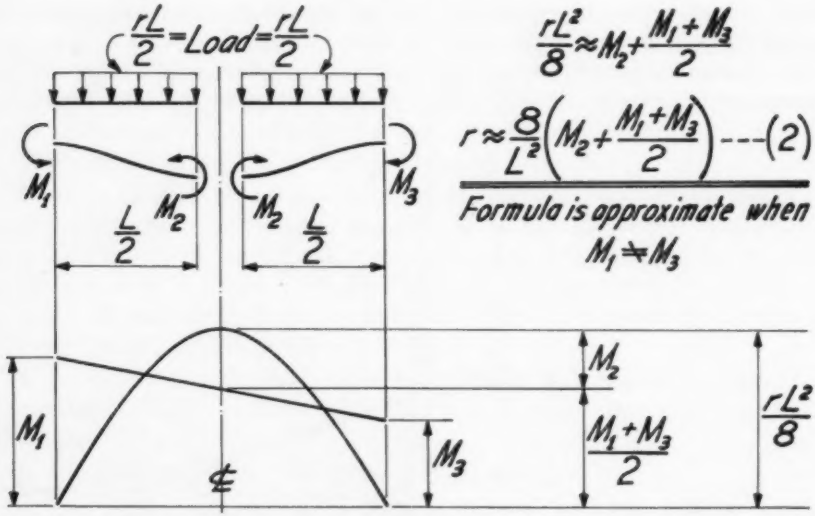


Fig. 13—Ultimate flexural resistance of uniformly loaded beam

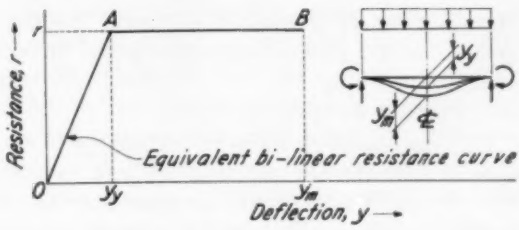


Fig. 14—Maximum ductility of reinforced concrete beam

$$\mu = \frac{y_m}{y_y} = \frac{\text{Collapse deflection}}{\text{Yield deflection}} = \frac{10}{\phi - \phi'} < 20 \quad \text{-----(3)}$$

If the static load-deflection curve can be drawn. Fig. 12 illustrates such a curve, identified by the line OCEB. For design purposes, this curve is replaced by the equivalent bi-linear curve, OAB, of the type previously shown in Fig. 5, chosen so that the total area under the equivalent curve equals that under the original curve. This leaves the strain energy absorption at maximum deflection unchanged, largely eliminating any error in making the substitution.

The properties of interest are those of the equivalent curve, and include the maximum ordinate, r, yield deflection, y_y, and maximum deflection, y_m. Of these properties, the most important is the yield resistance, r. The value of the yield resistance in flexure is considered to be reached when localized plastic deformation of the reinforcing steel occurs at locations sufficient in number to cause collapse. As a simple example, Fig. 13, the ultimate flexural resistance of a uniformly loaded beam with end restraints would be attained when yield moments are developed at each end and at midspan.

Similar collapse considerations apply in determining flexural resistance of slabs, in which yielding is assumed to be concentrated in

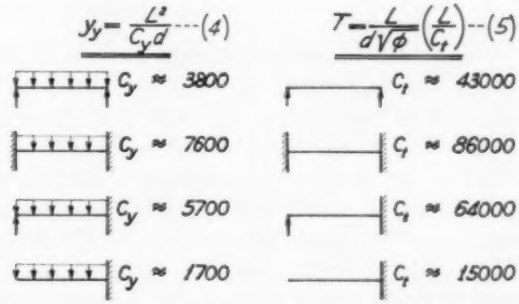


Fig. 15—Yield deflection, y_y, and period of vibration, T

(All units of length are in inches)

"crack-lines" of a certain pattern. The theory and application of the yield-line and yield-hinge hypotheses have been presented in previous ACI literature by Hognestad,[9] and by Whitney, Anderson, and Cohen,[10] and will not be included here.

It is necessary to have an indication of the amount of ductility available in a reinforced concrete flexural member. The expression shown as Eq. (3) in Fig. 14 gives the approximate collapse deflection of a rectangular beam or one way slab in terms of the yield point deflection.[3,11] In this expression, ϕ is the percentage of tensile steel and ϕ' is the percentage of compression steel. The upper limit of μ is held to about 20, in the case of beams reinforced with intermediate grade steel, and, for shear failures, which are brittle in nature, μ is restricted to a value of about 2.

Where it is necessary to roughly estimate deflections, the effective yield deflection, y_y, based on the equivalent bi-linear resistance-deflection curve, can be approximated from an expression[11] of the form shown as Eq. (4) in Fig. 15, where C_y is a factor which depends on the yield point and span-wise distribution of the steel, end conditions, and type of member (one or two way slab, for example). Representative values for a one way slab with various end conditions, and reinforced with equal amounts of positive and negative steel of intermediate grade, are as shown. The deflection, y_m, at collapse is obtained as the product μy_y.

For approximating the period of vibration, T, which is the remaining structural property required, the rather complex expressions found in most texts are simplified to the form shown in Eq. (5) of this same figure. The values of C_t shown apply to one way slabs with equal amounts of positive and negative steel, and are based on the use of the equivalent bi-linear resistance-deflection curve of Fig. 5 and 12.

When a member carries dead load in addition to its own weight, T is increased in accordance with the fact that the period of vibration varies as the square root of the total mass involved. The period of vibration of an earth covered roof slab would be computed on this basis.

Design stresses

Structural resistance cannot be calculated without first assuming design stresses. These stresses are based for the most part on the yield strength of the reinforcing steel and the ultimate strength of the concrete. In the case of intermediate grade reinforcing steel, an elevated yield stress of 50 ksi is used for blast loading. This value is based on considerations which include the average variation of material properties above the specified minimum, and the increase in yield stress under rapid loading. Calculation of yield moment capacity of flexural members is based essentially on the provisions of the Appendix in the ACI Code dealing with ultimate strength design.[12]

The ultimate compressive strength of the concrete under blast loading is assumed to be one-third greater than the static value. For 3000 psi coral concrete, for example, 4000 psi is used.

The design of columns follows the recommendations in the Appendix of the ACI Code,[12] with elevated values being used for the ultimate strength of the concrete and for the yield strength of the steel.

The strength of concrete members in shear, which is currently a subject of considerable research, is especially important in the design of members subjected to high overpressures. In these cases, member depths are controlled by shear, and the use of stirrups may be necessary, even in slabs.

Until recently, evaluation of shear strength has, in general, followed conventional practice, except for the use of allowable stresses roughly double the usual basic values, and the assumption of a critical section at $d/2$ from the face of support. This procedure is now being revised in light of investigations[11,14] in this field which point to the significant influence of variables in addition to concrete compressive strength and percentage of web reinforcement, such as end conditions, span to depth ratio, and percentage of longitudinal steel.

The ultimate bond stress is assumed as $0.15\ f_c'$, which is slightly more than double the value for top bars specified in the ACI Code.[15] Minimum bond laps of 30 diameters are used.

A shear stress of $0.1\ f_c'$ is used for the cracking load in shear walls and diaphragms. The design of these elements is based largely on results of tests reported by Benjamin and Williams.[16]

Safety factor

The ultimate design approach requires the use of a safety factor, which is incorporated by multiplying the anticipated overpressure by a number which, in general, ranges from 1.0 to 1.5, depending on the use of the station and the overpressure level.

EXAMPLE

The following example applies the foregoing criteria to the investigation of the roof slab of a buried structure. The slab has a clear span of 20 ft and carries 4 ft of earth cover. Total depth is 22 in. and effective depth is 20 in. One percent tensile steel is used at supports and at midspan. Compression steel percentage is 0.25. Concrete static ultimate strength is 3 ksi and reinforcing steel is intermediate grade. End conditions permit the development of yield moments at the supports.

The problem is to determine the peak overpressure which will cause failure in flexure. The overpressure is assumed to decay linearly from its peak to zero in 0.30 seconds.

Solution

$\phi = 1.0$, $\phi' = 0.25$, $d = 20$ in., $L = 20$ ft $= 240$ in., $t_1 = 0.30$ sec.

Dead Load (psi)

$$\text{Slab} = W_c = \frac{22}{12} \cdot \frac{150}{144} = 1.9$$

$$\text{Earth} = W_e = \frac{4(100)}{144} = 2.8$$

$$\text{Total} = W' = 4.7 \text{ psi}$$

Ultimate moment capacity

From Eq. (A1), Ref. 12, (neglecting compression steel with minor error)

$$q = p \frac{f_y}{f_c'} = (0.01) \frac{50{,}000}{1.33(3{,}000)} = 0.125$$

$$M_u = bd^2 f'_c \, q(1 - 0.59q) = (1)(20)^2(1.33)(3{,}000)(0.125)[1 - (0.59)(0.125)]$$

$$M_u = 185{,}200 \text{ in. lb per in. of width}$$

Flexural resistance

From Eq. (2), Fig. 13

$$\frac{8}{L^2}\left(M_s + \frac{M_1 + M_2}{2}\right) = \frac{8}{L^2}(2M_u) = \frac{16M_u}{L^2}$$

$$= \frac{16(185{,}200)}{(240)(240)} = 51.4 \text{ lb per in. per in. width} \quad \ldots\ldots 51.4 \text{ psi}$$

$$\text{Net available } r = 51.4 - 4.7 \quad \ldots\ldots 46.7 \text{ psi}$$

Maximum ductility

From Eq. (3), Fig. 14

$$\mu = \frac{10}{\phi - \phi'} = \frac{10}{1 - 0.25} = 13.3$$

Period of vibration

From Eq. (5), Fig. 15

$$\frac{L}{d\sqrt{\phi}} \frac{L}{C_t} = \frac{240}{20\sqrt{1}}\left(\frac{240}{86{,}000}\right) = \quad \ldots\ldots 0.0335 \text{ sec}$$

$$\text{Mass correction} = \sqrt{\frac{W'}{W_c}} = \sqrt{\frac{4.7}{1.9}} = 1.58$$

$$\therefore T = (1.58)(0.0335) = \quad \ldots\ldots 0.053 \text{ sec}$$

Overpressure to Cause Collapse

$$\frac{t_1}{T} = \frac{0.30}{0.053} = 5.7 \text{ and } \frac{y_m}{y_y} = \mu = 13.3$$

From Fig. 6,

$$\frac{p_m}{r} = \frac{p_m}{46.7} = 1.16$$

$$p_m = 1.16(46.7) = 54.2, \text{ say } 54 \text{ psi Ans.}$$

The effect of changing various parameters is easily studied by means of a response chart, such as Fig. 6. For example, to estimate the value of p_m at which the slab will remain completely elastic under a pulse having $t_1 = 0.30$ sec., use $y_m/y_y = 1.0$ and $t_1/T = 5.7$. From Fig. 6 it is found that $p_m/r = 0.5$, from which $p_m = 23.3$ psi. This is about 43 percent of the overpressure causing collapse. Similarly, if y_m/y_y is limited to 5, with other conditions remaining as in the original problem, p_m is found to be about 47 psi, or about 87 percent of the overpressure causing collapse.

To find the duration, t_1, at which the slab will remain elastic under a pulse having $p_m = 54$ psi, use $y_m/y_y = 1$ and $p_m/r = 1.16$. From Fig. 6 it is found that $t_1/T = 0.32$, giving $t_1 = 0.017$ sec., which is about 6 percent of the t_1 value at collapse.

DESIGN DETAILS

Experience has shown that special attention to avoid deficiencies in connections, joints, and other details is even more necessary in blast resistant construction than in conventional construction, in order to develop the full strength of structural elements.[17] The reliability of details critical in shear can be improved by providing shear bars to carry a large percentage of the total load, which may alleviate the brittle characteristics of this type of failure. Generous bar laps are desirable, and special anchorage should be used when bond stresses become excessive. There is some question regarding the dynamic strength of lapped splices, which indicates the desirability of locating them at points of low stress, or else substituting welded splices. The possibility of stress reversal caused by elastic rebound or negative overpressure dictates that a part of the negative steel at supports be made continuous. For the same reason, vertical stirrups rather than bent bars are preferable for web reinforcement.

CONCLUDING REMARKS

The conventional working stress approach will produce structures having safety factors against collapse in the various modes of failure which are not uniform, being too high in some cases and too low in others. The objective of maintaining a consistent amount of reserve strength in each failure mode can only be attained by design procedures which evaluate ultimate strength and consider the transient nature of the loading. The procedures currently used approach this goal, involve simple extensions of routine procedures which are easily learned by the average designer, and are adaptable to conditions requiring a maximum amount of output in a very short time.

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CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

SLA Translation Center's Role in Engineering Technology

By DONALD W. RAMSDELL*

In the last two decades, the startling growth of our technological civilization has been apparent on a world wide scale. The emergence of the United States and the USSR as the two present-day great powers has stimulated competition between them. But recently both countries have come to realize that duplication of research is costly both in terms of time and money. Russia has become more amenable to allowing the results of her research to be disseminated in other countries. However, since few engineers and research scientists are familiar with the Russian language, most of this work must be translated.

Though the language barrier is not as insurmountable with German, French, Italian, Spanish, etc., and we have always exchanged ideas much more freely with these western countries than with Russia, it is easily understood that one can work more competently and quickly with his own language, rather than a foreign tongue. It was to supply this need that the Special Libraries Association's Translation Center was founded in 1953 at The John Crerar Library in Chicago. Until 1959, it operated as the only such center in the United States, collecting translations published by both private and governmental agencies, foreign and domestic. These were indexed in a publication entitled *Translation Monthly* which was available to the public on a subscription basis.

In January, 1959, the Office of Technical Services of the U. S. Department of Commerce established a new center for translations. The OTS now serves as the collection center for governmental agencies, both domestic and foreign, while SLA continues to collect nongovernmental translations, both domestic and foreign.

Copies of all the translations received by both agencies are deposited at the SLA Translation Center and the Library of Congress, where they are available in photoprint or microfilm copies. OTS is responsible for the cataloging and abstracting. Card catalogs filed by author, title, sub-

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*Chief, Special Libraries Association Translation Center, The John Crerar Library, Chicago, Ill.

ject, serial number, and SLA translation number are maintained by both institutions. Prices charged by both are identical.

These translations are indexed in a semi-monthly publication issued by OTS entitled *Technical Translations*. This has superseded SLA's *Translation Monthly* which ceased publication in December, 1958. *Technical Translations* lists translations obtained from all sources, both governmental and private; translations available from commercial firms; translation work in progress in government agencies; and a list of periodicals which are completely translated. It is arranged in broad subject categories, giving complete bibliographical information, price, where obtainable, and usually a brief abstract. The cost is $12 per year for a domestic subscription, and $16 for a foreign subscription. Orders for *Technical Translations* should be sent to the Office of Technical Services, U. S. Department of Commerce, Washington 25, D. C.

The SLA Translation Center collects an average of 5000 translations yearly, and now has holdings of approximately 35,000 publications, of which 45 percent are Russian and 55 percent are from other languages. However, since it is a cooperative undertaking, its growth depends on contributions from outside organizations. If it is to fulfill its aim of becoming a national repository of hitherto unpublished scientific and technological translations, and making them available to requesters, it must have the cooperation of all who are engaged in any kind of a translation program.

Since many organizations do not wish to be disclosed as the originator of a translation, the SLA Center has agreed to obliterate all identifying information before releasing any publication for general use.

It is hoped that the Center will grow in a geometric progression as its aims and resources become more widely known.

Bridges

Flyover built into London road junction

Engineering (London), V. 188, No. 4877, Oct. 9, 1959 pp. 314-315

Reviewed by Abon L. Mirsky

London's Chiswick Flyover carries the Cromwell Road extension (London's new western approach) to the Great West Road over the junction of four main roads, by means of a four-span overpass structure over a 400 ft diameter traffic circle, an approach bridge, and slip ramps. Superstructure is formed of precast prestressed inverted-T units with in-situ deck; but abutments and retaining walls are massive, of mass concrete faced with 1½

Where the English title only is given in a review, the book or article reviewed is in English. If it is followed by a foreign title the work reviewed is in that language. In those cases where the foreign title cannot conveniently be set in type or is not available, the language of the original article is indicated in parentheses following the English title. Copies of articles or books reviewed are not available through ACI. Available addresses of publishers are listed in the June "Current Reviews" each year. In most cases ACI can furnish addresses of publications added later.

For those members who cut apart this section for pasting on cards for card indexes, a limited number of complimentary reprints of the "Current Reviews" section are available from ACI headquarters on request.

million hand-finished brick and cast-in-place coping. All substructure elements except one pier are founded on reinforced concrete rafts.

Design has been severely criticized, especially by the contractor, for lacking elegance, being structurally old fashioned, taking too long to build, and being too costly. (Contractor's statement is printed **verbatim** in *The Engineer* (London), V. 208, No. 5411, Oct. 9, 1959, p. 397. Defense has been equally spirited: for a statement by the consulting engineers, see *The Engineer*, Oct. 16, 1959, p. 439, or *Engineering*, Oct. 16, 1959, pp. 335-336. Additional comments, pro and con, from various sources appear in subsequent issues.)*

*A similar controversy is also raging around the Bell Weir Bridge at Staines (*The Engineer*, V. 208, 1959: Oct. 23, p. 483; Nov. 6, p. 567; Nov. 13, p. 607).

Critical survey of bridge design

O. A. Kerensky, *Engineering* (London), V. 188, No. 4873, Sept. 11, 1959, pp. 180-184.
Reviewed by Aron L. Mirsky

This paper, read before the Engineering Section of the British Association for the Advancement of Science at York in September, 1959, under the title "Bridges—a Survey," covers many aspects of bridge design: early bridge history, recent developments in type and materials, loadings, standardization, and competitive designs. Recommended reading, for administrators as well as engineers.

Prestressed bridge is redesigned in midstream

Engineering News-Record, V. 163, No. 24, Dec. 10, 1959, pp. 34-36

The Tugela River highway bridge in Natal, South Africa, originally designed as a continuous prestressed concrete structure over its entire length, was redesigned during construction following a disastrous and unseasonable flood. The result is a combination of two different sections, one as a continuous prestressed length of about 955 ft, the other a continuous prestressed two-span structure about 395 ft long.

Jean de Lattre de Tassigny Bridge and its three level interchange on the right bank (Le Pont Jean de Lattre de Tassigny et son carrefour a trois niveaux rive droite)

C. Roques, A. Thiebault, P. Xercavins, A. Duclot, M. Blanchet, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 138, June 1959, pp. 633-655
Reviewed by Henri Perrin

After a general description of the arterial facilities in the city of Lyon, a summary of the design is presented along with detailed information about the erection and fabrication of the bridge.

Bridge and embankment over Sio Sound

Anker Engelund, *Ingenioren* (Copenhagen), International Edition, V. 3, No. 4, Dec. 1959, pp. 113-118
Reviewed by Jesper Strandgaard

Describes briefly the design and construction of a prestressed concrete bridge, consisting of 20 equal, simply supported 27-m spans on concrete piers. Both the cellular piers and the complete box girder spans, less paving, were floated in place on pontoons capable of carrying up to 300 tons. Only bottom reinforcement was prestressed (Dywidag System); main bars were Tentor steel while secondary reinforcement was ordinary smooth bars.

New Melling Bridge, New Zealand

A. N. Grigg and E. R. Ireland, *New Zealand Engineering* (Wellington), V. 14, No. 7, July 15, 1959, pp. 209-221
Highway Research Abstracts Jan. 1960

The bridge, of five 90-ft spans with a clear road width of 30 ft, crosses the Hutt River which is subject to severe "flash-floods."

The design adopted called for composite action between the precast prestressed main beams and the concrete deck. The deck was cast with the main beams "propped" at midspan by a controlled elastic vertical reaction which was continuously adjusted during the casting of each slab so as to maintain a condition of zero bending moment over the temporary midspan prop.

Design considerations are discussed in Part 1, and the method used to determine cable eccentricity is described.

Part 2 of the paper describes the construction process with particular emphasis on changes which were made in the "design-stage" plan of operations so as to suit the contractor, and discusses the advantages and disadvantages of precasting each beam in a series of "blocks", as was done, instead of as a complete unit as was originally intended.

Construction

Water tower for the town of Uppsala (in Swedish)

I. GULLSTROM, L. GERDE, B. BANG, and G. GRANDELL, *Cement och Betong* (Malmo), V. 33, No. 4, Dec. 1958, pp. 203-216
Reviewed by BENGT F. FRIBERG

A truncated, cone shaped, high-level water reservoir provides a primary storage of 1,600,000 gal. and an additional storage, at less head, of 1,000,000 gal. for fire reserve. The reservoir has a base diameter of 47 ft, a top diameter of 123 ft, and height of 95 ft, with a central 18.5-ft circular shaft for the stairs and elevator, which go to the sight-seeing floor at the top. They are supported by precast and prestressed beams which in turn are freely supported from the central shaft to the outside ring. The plan view is a 20-sided polygon both inside and out with varying wall thickness, prestressed circumferentially with 66-, 113-, and 200-kip post-tensioning cables overlapping $\frac{1}{20}$th of the circle at diametrically opposed points.

The central shaft was built first using sliding forms, and supported a construction crane. The dimensions of the outside wall were held to close tolerance by six, 20-sided steel rings, supported by 20 sloping steel supports, and with radial prestressed ties to the central shaft which braced the forms in addition to the conventional scaffolding. Inside, wood forms were installed as concrete placing proceeded continuously around the wall in 5-in. lifts, 1140 cu yd being placed and vibrated in 216 hr. Maximum form deviation from exact position was less than 0.10 in. The circumferential cables were prestressed in steps, 30 percent prestress was applied at 10 days and full prestress before 90 days.

Development in overhead electrification of railways as it affects the civil engineer

RALPH E. SADLER, *Proceedings*, Institution of Civil Engineers (London), V. 12 (Session 1958-59), Feb. 1959, pp. 125-156; discussion, Aug. 1959, pp. 631-651
Reviewed by ARON L. MIRSKY

Discusses a few of the matters which have arisen in conjunction with the recent surge of railroad electrification in England: design of partially prestressed concrete bridge decks, demolition and modification of masonry arches [via stripping them of the lower arch rings, or reconstructing the central (crown) section], etc. French practices in this connection, which served as guide for the British work, are also noted.

Current concrete construction activity in a Swedish town (in Swedish)

C. A. LOCKERT, *Cement och Betong* (Malmo), V. 34, No. 2, June 1959, pp. 137-144
Reviewed by BENGT F. FRIBERG

Describes the active concrete building construction under way in Sweden in an industrial town with a population of approximately 80,000 (Vasteras).

An 11-story apartment house with 132 apartments employs precast wall panels of room-wall size (including openings, even windows), precast columns, stairs, complete bathrooms, balcony floors, and solid railings. The lower portions of the floor slabs are precast, with negative reinforcement, while the top portions are placed in the field. Temporary walls covered with tarpaulin protects the construction during erection and are moved up a story at a time. The building elevators are used for construction purposes; the machinery is placed in the basement with the cable sheaves being advanced with the construction.

A 1650 ft long four-story apartment house is going up with floors and walls being cast in large wall size steel forms; corner and door frames are set in before concreting.

A new town hall with an 18-story carillon tower employs a concrete frame with some precast elements. More than 100 basementless, one-family houses on concrete pedestals used precast perimeter beams, the rest of the construction being wood.

A ten-story office building with 350,000 sq ft of floor area, for the town's largest industry was designed on a 40-in. module, with 3- and 6-module column spacing. Columns, floor joists, facing panel supporting studs, and sill beams are precast to tolerances exact enough to fit window frames and metal wall panels.

Low-cost fallout shelter

Concrete Construction, V. 5, No. 2, Feb. 1960, pp. 32-33

A low-cost reinforced concrete fallout shelter has been incorporated as integral elements in 40 new homes under construction in a suburb of Denver. Cast as a part of the foundation structure, the 12-in. concrete ceiling of the shelter forms about one-half of the garage floor. The shelter's vertical walls are 8 in. thick and reinforced; three sides of the shelter are surrounded by earth, since it is below grade level; the other wall faces the remainder of the basement.

First mechanized post office has unusual barrel-shell roof

Engineering News-Record, V. 164, No. 2, Jan. 14, 1960, pp. 30-32, 34

Roof was selected to provide large expanses of unobstructed floor space for electronically controlled mail-flow system. It also eliminates exterior buttresses of conventional arches which would have interfered with outside truck traffic. Roofing a 420 x 300-ft area, the intersecting barrels are separated by 2-in. expansion joints. Only two groups of interior columns are required. Both exterior and interior columns are designed as fixed-base cantilevers, hinged at the top and free to deflect laterally. The 6-in. shells require about 800 cu yd of concrete in each unit. Tensioned horizontal tie beams balance the thrusts of the edge beams.

Concrete specifications required high-early strength of 4000 psi at 7 days, a density of 110 lb per cu ft, and a 2-in. slump. The aggregate used was sand and expanded shale. The only admixture used was an air-entraining agent.

Construction of the Exhibition Palace of the Centre National des Industries et Techniques (La construction du Palais des Expositions du Centre National des Industries et Techniques au rond-point de la Defense)

Annales, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 137, May 1959, pp. 475-519

Conception and erection of the vaulted shell (Conception et execution de la voute-coque)

N. ESQUILLAN

Realizations, design, and tests (Realisations, calculs, et essais)

G. LACOMBE, P. FAESSEL, and R. PERZO

Reviewed by HENRI PERRIN

The now well known exhibition center in Paris is described by its designers. The novelty resides in the report of tests conducted on small concrete shells and slabs to check their buckling strength. [See also Current Reviews, ACI JOURNAL, V. 31, No. 1, July 1959 (*Proceedings* V. 56), pp. 72-73.]

Oil tank of concrete in northern Sweden (in Swedish)

G. ENSKOG, *Cement och Betong* (Malmo), V. 33, No. 3, Sept. 1958, pp. 131-137

Reviewed by BENGT F. FRIBERG

Describes the construction of a circular, post-tensioned fuel oil tank having a 100 ft diameter, 46.5 ft height, and a capacity of 2,650,000 gal. A dome-shaped roof of steel served as the working platform and for bracing of the slip forms of the 9.9-in. walls.

It also permitted heating of the interior during construction. The walls were post-tensioned vertically with 1.03-in. alloy rods, 37.5 in. center to center, and horizontally in two layers, with 0.198-in. wire applied with a Swiss automatic wrapping machine. The inside was given several coats of epoxy-resin paint and reinforced with glass fiber. The floor has circular, edge cable prestressing, and extends beyond the walls; it was placed on a 32-in. sand cushion.

Construction Techniques

Pacesetting plant stresses automation, HMS

RALPH S. TORGERSON, *Rock Products*, V. 63, No. 2, Feb. 1960, pp. 98-103

This sand and gravel installation has a capacity of 600 tons per hr to the surge pile and about 400 tons per hr through screening, washing, and crushing operations. Two outstanding features are extensive use of electronic controls and heavy-media flotation system to upgrade aggregates.

Pneumatically applied concrete in tunnel construction (in Serbian-Croatian)

S. HADZOVIC, *Nase Gradevinarstvo* (Belgrade), V. 13, No. 5, May 1959, pp. 108-112
Reviewed by J. J. POLIVKA

Gives general properties of shotcrete and good practices in mixing and placing. Describes equipment used in Yugoslav tunnel construction, with references to experiences in other countries, especially in United States and France. Speed of construction and its economy is emphasized.

Prefab forms permit fast pours

MIKE SPRONCK, *Construction Equipment*, V. 21, No. 2, Feb. 1960, pp. 35-38

There is hardly a spot in its near-mile length where two dimensions are the same in this expressway retaining wall. The wall contains three principal cross sections and ranges in height from 4 to 32 ft, most over 25 ft. The wall tapers from top to bottom and, in some areas, along its length too. Footings remain horizonal, so the sections stair-step down the slight grade.

To speed placement of reinforcement, the contractor built racks which spanned the wall base to serve as spacing and tying jigs. Prefabricated steel forms were used throughout.

The design called for a 6-in. rubber waterstop joint between placements. The joints are open to a point 4 in. back of the face. In forming the joint, ½ x 4-in. Styrofoam strips were used instead of the conventional wood strips.

Dams

Control of cracking in concrete gravity dams

WILLIAM R. WAUGH and JAMES A. RHODES, *Proceedings*, ASCE, V. 85, PO 5, Oct. 1959, pp. 1-20
AUTHORS' SUMMARY

The fundamental cause of temperature cracking in concrete gravity dams is the restraint which accompanies temperature changes. Paper traces evolution of crack control measures (cement factor, cement replacement material, concrete placing temperature, lift thickness, rate of placement, and insulation), and describes current Corps of Engineers practices and results.

Hydroelectric harnessing of the Posterior Rhine (L'amenagement hydroelectrique du Rhin Posterieur)

OSKAR SPÖRRI, *Le Génie Civil* (Paris), V. 136, No. 7, Apr. 1, 1959, pp. 149-158
Reviewed by ARON L. MIRSKY

The Rhine River is formed in east Switzerland by the union of two small headstreams, the Hinter Rhein (Rhin Postérieur) and the Vorder Rhein. Project, on Swiss-Italian border but mainly in the Swiss canton of Grisons, required considerable persuasion and negotiation, and was finally started in 1956. It includes four main elements, the Val de Lei storage reservoir and three conduits. Article covers briefly the history and financing of the project, the geologic and hydrologic characteristics, and the various structures, with particular emphasis on the construction

of the Val de Lei dam, a doubly-curved arch structure with a maximum height of 138 m, maximum thickness of 28 m, and crest length of 635 m, and requiring a total of 840,000 cu m of concrete. Design was by the trial-load method, using an electronic computer, and was checked using a 1:66 model. *

*More details will be found in a series of articles in *Schweizerische Bauzeitung* (Zürich), V. 77, 1959: No. 26, June 25, pp. 405-409 (L. Kalt and Niklaus Schnitter); No. 30, July 23, pp. 479-484 (Klaus Wenzel); No. 34, Aug. 20, pp. 543-546 (R. Härri); and No. 41, Oct. 8, pp. 675-679 (Niklaus Schnitter).

Design

New composite-beam concepts

FRANK D. STEINER, *Civil Engineering*, V. 30, No. 2, 1960, pp. 72-74

Briefly reviews graphical methods for solving some of the problems in composite beam construction. It also discusses advantages of this type of construction and describes steel-plate armoured concrete structures and prestressed composite beams. It describes some of the advantages which can be gained by utilizing the materials which are often available, or actually present, in the construction.

Reinforced concrete ultimate strength design: Actual aspect of the question (Le calcul du beton arme a la rupture: Etat actuel de la question)

R. CHAMBAUD, *Annales*, Institut Technique du Bâtiment et des Travaux Publics (Paris), No. 135-136, Mar.-Apr. 1959, pp. 307-351
Reviewed by HENRI PERRIN

Since the author's paper on rupture by bending and shear [Current Reviews, ACI JOURNAL, No. 3, V. 30, Sept. 1958 (*Proceedings* V. 55), p. 406], numerous tests have been conducted and the most important of them are now reported: mechanical properties for cold worked steel and ultimate flexural, compressive, and shear strengths for concrete.

A discussion of every element involved in the determination of the safety factors closes the technical report. In conclusion, the author reviews the studies done in France and abroad as well as the controversies going on about ultimate strength design.

Simplified design of reinforced concrete members under eccentric loads (in Slovenian)

S. TURK, *Gradbeni Vestnik* (Ljubljana), V. 10, No. 61-64, 1958-1959, pp. 33-40
Reviewed by J. J. POLIVKA

Another article on author's simplified design method for concrete members with double reinforcement and subject to eccentric loads resulting in combined compressive and bending stresses. Consideration of buckling is treated in a new way. Nonrectangular and other types of irregular sections are also considered.

Influence line ordinates at the 10th points of continuous beams (Zehnteilige Einflusslinien fuer durchlaufende Traeger, Band I)

GEORG ANGER, Verlag von Wilhelm Ernst und Sohn, Berlin, V. 1, 1958, 272 pp., 41 DM
Reviewed by FRITZ KRAMRISCH

V. 1 is the last of the three-volume set to appear on the market. V. 3, which was prepared primarily for the use of structural engineers, was reviewed in Current Reviews, ACI JOURNAL, V. 28, No. 6, Dec. 1956 (*Proceedings* V. 53), p. 614, and V. 2, which was prepared for the bridge designers, was reviewed in ACI JOURNAL, V. 31, No. 1, July 1959 (*Proceedings* V. 56), p. 77.

V. 1 furnishes the basic load values for all kinds of loading conditions. Also given are formulas for the calculation of moments and shears for continuous beams up to six bays, having any ratio of spans and type of end conditions. Many fully worked out examples are included in the text.

Because of its basic character it is quite important to use the book with the other two volumes. It contains a wealth of information, readily worked out formulas for practically any possible combination of loading arrangements, different span ratios and all

types of end conditions. But due to this versatility, and the intention to cover "everything," it is rather involved and not too easily handled by somebody who does not know enough German to follow the explanations and directions of the text.

Materials

Radioactive rays control clinker depth

D. H. Gieskieng, *Rock Products*, V. 63, No. 2, Feb. 1960, pp. 150, 154-158, 166, 170

The gamma-ray gage, bed-depth control marks a significant advance in the automation of shaking-type clinker coolers in cement plants. Purpose of the gage is twofold: to guard the cooler against sudden overloading and to maintain operation near maximum stable bed depth. Temperature stability, clinker size, temperature measurement, and control apparatus are discussed.

The coming role of pozzolans. Part IV—Product potential through process design

Wolf G. Bauer, *Pit and Quarry*, V. 52, No. 7, Jan. 1960, pp. 181-182, 184

Basic and applied phases of pozzolan development are discussed. A basic processing study takes into account such factors as upgrading, heat activity, reactive surface development, particle sizing and gradation, and possible use of certain compatible accelerators or catalysts. Results of such research serve as guideposts to process design with pilot equipment. [See also Current Reviews, ACI Journal, V. 31, No. 10, Apr. 1960 (*Proceedings* V. 56), p. 1084.]

Epoxy-based masonry materials for construction, repair, and maintenance

Philip Maslow, *The Construction Specifier*, V. 12, No. 5, Feb. 1960, pp. 24-30

Discusses the development of synthetic resins in general and epoxy resins in particular. It describes the various tests performed on samples and lists the physical properties. Since epoxy compounds are commonly used on floors, this use under various conditions is described and also the resistance to particular chemicals is classified. The use for highway surfacings and in repair and restoration of concrete or masonry structures is discussed with the advantageous qualities enumerated.

Pavements

Concrete roads (Chaussées en béton de ciment)

Circulaire No. 2, Ministere des Travaux Publics, des Transports et du Tourisme, Paris, 1958, 110 pp., 420 Fr.

Road Abstracts Nov. 1959

Instructions are given regarding materials and construction methods which may be used for concrete surfacings on roads and runways in France and conditions to be observed by the contractor.

Prestressed concrete surfacings (in German)

G. Streit, *Baumaschinen und Bautechnik*, V. 6, No. 5, 1959, pp. 161-166

Road Abstracts Nov. 1959

This review discusses the following topics: purpose and advantages of prestressing surfacings, special characteristics of prestressed concrete surfacings, prestressing methods, prestressing forces, prestressing elements, slab thickness, slab length, joints, friction between surfacing and subgrade.

Prestressed Concrete

A theory to predict the neutral surface position in rectangular prestressed concrete beams as cracking develops

H. J. Brettle, *Proceedings*, Second National Prestressed Concrete Conference, Wellington, New Zealand, 1958, pp. 59-65

Author's Summary

A theory for predicting the neutral surface position in rectangular prestressed concrete beams is proposed. The theory is mainly concerned with

conditions after cracking has commenced. The theoretical relation between applied bending moment and neutral surface position is substantiated by the observed results from 26 beams of dimensions 4 in. x 6 in. x 10 ft. The beam factors were varied over a large range, well beyond that expected in practice.

Strength of prestressed concrete continuous beams and simple plane frames

P. B. Morice and H. E. Lewis, *Proceedings of a Symposium on the Strength of Concrete Structures*, Cement and Concrete Association (London), 1958, pp. 377-400

Authors' Summary

The paper provides an analysis of prestressed concrete continuous beams and simple plane frames in their ultimate load condition. The invariance of elastic design stresses under linear tendon transformations is briefly shown and it is demonstrated that the same is theoretically true for the ultimate load condition.

Tests on a number of beams and frames are described and the results are given, showing that the theoretical propositions apply to the structures tested.

A comparison is drawn between these tests and those of Macchi. Significant differences between the tests lead to some suggestions for further research in this field.

Strength of statically indeterminate prestressed concrete structures

Y. Guyon, *Proceedings of a Symposium on the Strength of Concrete Structures*, Cement and Concrete Association (London), 1958, pp. 305-376

Author's Summary

Calculations of the failing loads of statically indeterminate prestressed concrete structures are based on the consideration of plastic hinges, at which the moments are assumed to be known and equal to the resisting moments of these sections.

In a system with n redundancies, n plastic hinges are possible, and these make the system determinate for any load exceeding that at which the nth hinge appears. At a certain value of this load, an $(n + 1)$th hinge occurs and failure ensues; hence the safety factor.

Two general methods may be used to determine the ultimate load: one is based on the use of the bending moment diagrams and the enveloping curves (which are the diagrams of resisting moments); the other is based on the lines of thrust in the structure, and their limit lines (which are the loci of the extreme possible positions of the lines of thrust).

Full moment redistribution implies $n + 1$ points of contact between the bending moment diagram and the envelope (first method) or between the line of thrust and the limit line (second method).

Examples of calculations are given in the paper, and a comparison is made between experiment and theory. For this comparison, it is necessary to calculate the resisting moments. A chart is given which facilitates the evaluation of the resisting moments under the conditions considered, these being appreciably different from those in statically determinate structures.

An attempt has been made to check the possibility of full redistribution, based on an extrapolation into the plastic phase of the relations between the curvatures along the beam. This method, which will need to be revised when more experimental data are available,

Note

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has been applied to some of the data in the paper.

It is concluded that elastic calculations are still necessary, and in cases where, under normal load, tensile stresses would exceed certain limits, extra untensioned steel should be provided. The amount should be determined by the usual rules applied to reinforced concrete, the cables being a part of the reinforcement, provided the bond is effective.

Construction of a precast prestressed concrete factory in Northern Ireland

C. F. Candy, *Proceedings*, Second National Prestressed Concrete Conference, Wellington, New Zealand, 1958, pp. 84-93

Author's Summary

This paper confines attention to the construction of the prestressed members of this factory—40 box girders of 102 ft clear span and eight portals of 66 ft clear span—and gives a general description of the design of the members, the prestressing technique used, the problems encountered, and how these problems were overcome. It concludes with some general remarks on site organization.

Strength of prestressed concrete members

C. P. Siess, *Proceedings of a Symposium on the Strength of Concrete Structures*, Cement and Concrete Association (London), 1958, pp. 123-157

Author's Summary

Reviews the available knowledge regarding ultimate strength of prestressed concrete beams. Failures in both flexure and shear are considered but by far the greater amount of space is devoted to the former because of the greater detail in which this problem has been studied. Emphasis has been placed on the variables which effect the strength and mode of failure of beams.

The problem of determining ultimate strength has been approached in terms of the ultimate moment-carrying capacity, which has been related, for a given beam, to the steel stress at failure. This stress is in turn a function of the steel strain at failure and the stress-strain characteristics of the reinforcement. The problem is then reduced to one of determining the strain at failure as a function of the prestress and the properties of the cross section. For convenience in picturing the effects of various factors on ultimate strength, the steel strain at failure has been divided into two parts: the strain due to prestress and the additional strain produced by load. Simple examples are presented to illustrate the utility of this approach in visualizing the relative effects of the several variables considered.

Properties of Concrete

Air permeability of concrete (in Japanese)

Toyofujimaru Yoshii, Hitoaki Mori, and Mamoru Kanda, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 339-343

Ceramic Abstracts Feb. 1960

The air permeability of concrete is decreased by mixing a proper amount of pozzolan or blast furnace slag with the cement or by adding air-entraining agents. No consistent relations were found between air and water permeability.

Concrete and fire (Betong och brand)

Svenska Tarifföreningen, Stockholm, 1959, 112 pp.

Author's Summary

In 1956 a fire damaged a building in Gothenburg, Sweden, built with walls and roof of lightweight concrete, bearing construction of reinforced concrete, and roof beams of prestressed concrete. The experience led to Swedish insurance companies, the Swedish Tariff Union, and producers of concrete products conducting an extensive investigation and tests of prestressed concrete beams and lightweight concrete slabs to determine the strength and properties of such building elements under fire exposure.

On reporting the test results, it proved advisable to extend the work

to a handbook of properties and general information concerning concrete and concrete structures with special reference to fire exposure. An introduction describes elementary theory of concrete design and construction. Different chapters are devoted to the mechanical, physical, and chemical effect of fire on reinforced concrete in general and on prestressed concrete and lightweight concrete specifically. It is shown how damage to buildings can be estimated and how to judge what repair measures may be necessary. Repairs to structures damaged in fires are described.

Mortar and concrete containing surface-active agents (in Japanese)

TAKAO SODA, NAOKICHI TADA, ATSUSHI MAEKAWA and ISAMU EGUCHI, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 283-291

CERAMIC ABSTRACTS Feb. 1960

To determine the effect of surface-active agents on the properties of mortar and concrete, tests were made on fresh and hardened specimens. The relation of particle size distribution of fine aggregate, type of agent, entrained air content, and number and size of air voids in hardened mortar and concrete determined by the microscopic method are shown.

Freezing and thawing resistance of air-entrained concrete made with portland blast furnace slag cement (in Japanese)

CHOKUI SAITO, ISAO OHARA, and TOSHITSUGU IWABUCHI, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 266-272

CERAMIC ABSTRACTS Feb. 1960

Twelve portland blast furnace slag cements containing 30, 40, and 50 percent slag were prepared from four portland cements and one slag. Tests were made on concretes cured in water at 20 C for 14 and 91 days, an air-entraining agent being used. The entrained air content of these concretes was about half that of concrete made with portland cement, and it decreased with increasing specific surface of the cements. Freezing and thawing resistance of the concretes decreased slightly with increasing slag content, but specimens made with cement containing clinker high in $3CaO \cdot SiO_2$ and cured in water for a long period before the test showed superior durability.

Observations on methods of concrete vibration (Observations sur les methodes de vibration des betons)

P. REBUT, *Revue des Materiaux* (Paris), No. 525, June 1959, pp. 151-158

Reviewed by PHILLIP L. MELVILLE

Conflicting results have been obtained with vibratory equipment. Performance is affected by rheology, amount of water, mixing, use of fines, floculation, thexotropy. A study of the rheology of vibrated concrete is recommended to select admixtures.

Influence of pozzolanic fly ashes on the permanency of portland cement concrete (Influence sur la perennite des betons de portland de l'emploi des cendres volantes pouzzolaniques)

J. CHAPPELLE, *Revue des Materiaux* (Paris), No. 521, Feb. 1959, pp. 39-44

Reviewed by PHILLIP L. MELVILLE

A series of tests on the durability of concrete with from 0 to 40 percent fly ash replacement through cycles of freezing and thawing. It was found that durability was improved especially by the use of ground fly ash.

Physical properties of structural quality lightweight aggregate concrete

T. R. JONES, JR., T. J. HIRSCH, and H. K. STEPHENSON, *Report* E 52-59, Texas Transportation Institute, Aug. 1959, 46 pp.

This is the final report of a comprehensive study of structural quality lightweight aggregate and lightweight aggregate concrete using aggregates produced in Texas.

The body of this report presents figures, tables, and discussions which represent the conclusions based on the comprehensive study involving 28 batches of concrete with a large

number of creep, shrinkage, and compressive strength specimens.

The scope of this paper is focused, to some extent, on the creep and shrinkage characteristics since other properties can be obtained from specific mix designs. The report summarizes the properties of lightweight concrete made with the expanded clay and shale aggregates produced in Texas. It also recommend the qualities and quantities of concrete constituents and required properties of the final product to be specified.

Stressed condition of sections of ferroconcrete components, emerging as the result of the shrinkage of the concrete (in Russian)

I. I. Ulitskii, *Novoe v Stroit. Tekhn.*, No. 11, 1957, pp. 77-99; *Referativnyi Zhurnal Mekhanika* (Moscow), No. 5, 1958, Rev. 6021

Applied Mechanics Reviews
Dec. 1959

A study is made of the creep of the concrete, the increase of the modulus of elastic deformation, and also of the duration of the shrinkage. The characteristic of the creep of concrete ϕ_t is recorded (according to Kelvin) in the form of $\phi_t = a\,[1 - \exp(-bt)]$. For modulus E_t and shrinkage a_t the following formulas are proposed

$$E_t = E(1 + \delta\phi_t), \qquad a_t = a\,\frac{\phi_t}{a}$$

where E is the initial modulus, a the limiting relative strinkage, t the time, a, b, and δ constants. Equations of equilibrium are derived for a section of a ferroconcrete beam with one axis of symmetry and double unsymmetrical reinforcement. Arising from the equations of joint action of the deformation of the concrete and reinforcement an integral equation was obtained and a solution given in the form

$$\sigma = \frac{aE}{a}[1 - \exp(-\delta\phi_t)]$$

where σ is the stress on the concrete at the center of gravity of the reinforcement and ξ is a value depending on the degree of reinforcement.

Air permeability of concrete (in Japanese)

Kozaburo Watanabe, Shizumaro Nakamura, and Shozo Suzukawa, *Semento Gijutsu Nenpo*, V. 12, 1958, pp. 343-347

Ceramic Abstracts
Feb. 1960

Fly ash was substituted for part of the cement of concrete mixes containing 270 to 350 kg of total cementing material per cu m, and the effect on the air permeability and neutralization was tested up to the age of 1 year. The degree of neutralization increases with decreasing proportions of total cementing material, i.e., with increasing air permeability. The use of fly ash has essentially no effect on the degree of neutralization.

Concrete curing with steam at atmospheric pressure (Le traitement thermique des betons sour vapeur a la pression atmospherique)

H. Manche, *Revue des Materiaux* (Paris), No. 523, Apr. 1959, pp. 97-105

Reviewed by Phillip L. Melville

Sundry effects of high temperature curing are reviewed and a test program outlined. As a first phase, concrete cubes were cured under water at 11 C for 6 days or steam cured at 60 and 80 C. It was found that lower final strengths resulted from steam curing.

Creep and physical and mechanical characteristics of concrete (in French)

R. Dutron, *Bulletin* No. 1, International Association of Testing and Research Laboratories for Materials and Structures (RILEM), Paris, 1959, pp. 52-54

Battelle Technical Review
Feb. 1960

Tests were carried out on two types of concrete, one having a higher shrinkage and a lower modulus of elasticity than the other. These concretes were cured in two atmospheres: water and air at 50 percent humidity. At the moment of loading, one-half of the test pieces were kept in their original curing medium, the other half being placed in opposite conditions (test pieces in the water transferred to air, and vice versa).

Influence of mixing and exposure at 40 C on the workability and strength of concrete (Influence du gachage et de l'exposition a 40 C sur la fluidite et le resistance du beton)

L. Blondiau, *Revue des Materiaux* (Paris), No. 525, June 1959, pp. 137-145; No. 526-527, July-Aug. 1959, pp. 180-187
Reviewed by Phillip L. Melville

Concrete mixes with different water-cement ratios were exposed for 30, 40, 50, and 60 min to 20, 40, and 45 C at 90, 23, and 70 percent relative humidity, respectively. Workability was measured by the Slanville method. It was found that slag cements lose less workability when exposed to high temperature and low humidity.

Reference frame for the study of fresh concrete (Cadre systemsiqua pour l'etude des betons frais)

I. Leviant, *Revue des Materiaux* (Paris), No. 522, Mar. 1959, pp. 63-71
Reviewed by Phillip L. Melville

A graphical means of determining the properties for a given concrete mix using given aggregates. The method is used to study aggregate gradations.

Control of concrete strength by the ultrasonic method (in Russian)

O. E. Pflaumer, *Avtom. Dorogi* (Moscow), V. 22, No. 1, 1959, pp. 11-12. *Road Abstracts*, V. 26, No. 5, May 1959, p. 106
Highway Research Abstracts
Jan. 1960

The ultrasonic control method was used during the construction of a reinforced concrete bridge across the Moscow River. To check the variables involved in concrete manufacture which affect the accuracy of ultrasonic readings, control test cubes were prepared at the factory from the same material under identical conditions and subjected to ultrasonic and compression tests. A comparison of the results obtained showed that concrete of different strengths can sometimes give the same ultrasonic reading. Concrete strength was determined from the average of nine readings of each of the precast sections tested in place. Further control tests were made on cubes taken from the completed bridge components. Examples are given of the type of information obtained and of the calibrated curves which relate ultrasonic data to those obtained in compression tests. The IKL-5 ultrasonic instrument was found to be suitable for dimensions of 4½ to 6½ ft and to have an accuracy of 0.5 μs. The results of the ultrasonic and compression tests did not differ by more than 6 to 10 percent.

Concrete with slag aggregate (in French)

Laitiers & Tarmacadam, V. 12, No. 11, 1958, pp. 51-60
Road Abstracts
Dec. 1959

Results are given of research on concrete containing blast-furnace slag. Tests were made of the compressive and flexural strength, capillarity, shrinkage and swelling, resistance to freezing, and modulus of deformation of the concrete.

Deformation and cracking of hardened cement paste when shrinkage is restrained

F. A. Blakey and R. K. Lewis, *Civil Engineering and Public Works Review* (London), V. 54: No. 635, May, 1959 pp. 577-579; No. 636, June, 1959, pp. 759-762

Describes the work done to obtain information on the extent to which tensile stresses are set up in the concrete paste in relation to shrinkage caused by drying. The elastic properties of aggregate were not part of the study. Specimens 1 x 1 in. were cast around metal rods or tubes. As shrinkage progressed the central rod or tube was compressed and a continuously increasing strain was recorded. When the cement paste cracked there was a sharp change in the strain reading and it was possible to read from the recording chart to within 3 min the time in which the change in strain and therefore the crack occurred. Because there was no temperature compensating gage the strain gage record was not used for strain readings but the actual contraction of the steel outer tube was meas-

ured in the dial gage comparator used for the free shrinkage measurements.

In another set of specimens the current through a strip of silver conductive paint on the side of each restrained bar was recorded. The circuit was broken when the cement paste cracked, and the time at which this occurred was read from the recording chart within 4 min.

This study of the cracking of cement paste in which shrinkage is restrained showed that the elastic strain at failure is not constant as appeared to be so for mortars over a limited period of time. The authors develop an equation for the utimate elastic strain in the specimen which holds for conditions during the first 200 hr. After this time the authors observed a fairly sharp drop in the utimate elastic strain, but insufficient data were collected to define the behavior or explain it.

Structural Research

Reinforced and unreinforced concrete walls (in Swedish)

L. E. Larsson, *Cement och Betong* (Malmo), V. 33, No. 2, June 1958, pp. 59-68
Reviewed by Bengt F. Friberg

Centrally applied vertical load tests and loads at the third-points of the thickness were made on 70 wall elements, 100 in. high, 3.2 to 5.9 in. thick, unreinforced, and reinforced with 0.4 percent vertical and horizontal steel (one-half near each surface, which is the least percentage for dimensioning as reinforced wall in Sweden). Concrete strength varied from 3400 to 4800 psi cube strength with duplicate reinforced and unreinforced elements cast from one batch. Steel yield point was 63,000 psi.

The reinforced walls consistently failed at lower loads than corresponding unreinforced walls. The strength decrease for the centrally loaded walls was from 2.7 to 22.7 percent, the average being 10.0 percent, and for the eccentrically loaded walls it was from 3.0 to 25.6 percent, the average being 13.8 percent. The investigation puts the need for vertical reinforcement in walls not subjected to flexure in doubt. Horizontal reinforcement, intended to tie vertical shrinkage cracks, has not been questioned.

Folded plate construction: An investigation of collapse conditions

A. R. Dykes, *The Structural Engineer* (London), V. 38, No. 2, Feb. 1960, pp. 60-67

Describes an investigation into the collapse conditions of small roof structures in reinforced concrete.

A brief outline of the experimental work is given with pictures of the various test setups. The results of this investigation show that although the range is restricted, the measure of agreement between theory and experiment is remarkable for tests which were intended to be mainly exploratory in nature, no high degree of control being exercised. The theoretical approach is in a simplified form, no account being taken of secondary effects.

The fracture line theory gives a simple means of predicting collapse form and ultimate load. Use of an "approximate" method for deflection calculations is satisfactory, and is to be preferred on account of the easier solution obtained.

Evaluation of the flexural rigidity of slabs

P. S. Balint, *Constructional Review* (Sydney), V. 32, No. 12, Dec. 1959, pp. 18-25

A method is proposed for the direct evaluation of the flexural rigidity of slabs using strain and deflection measurements, taken on simply supported slabs when tested in bending. This is based on the results of tests to investigate the behavior of prestressed concrete slabs on elastic foundation. One of the objects of the investigation was to compare theoretical bending moment values with experimental ones.

In comparing values of Young's modulus as obtained from the compression test with the values calculated from the flexure test the latter runs consistently higher with an average difference

of 14 percent. In cases where the flexural rigidity of slabs has to be found accurately it appears that this described method of loading will give more appropriate results than would be obtained by testing the material.

Strength of concrete members in combined bending and torsion

STEWART ARMSTRONG, *Proceedings of a Symposium on the Strength of Concrete Structures*, Cement and Concrete Association (London), 1958, pp. 101-122

AUTHOR'S SUMMARY

Presents a review of existing theories and experimental evidence on torsion concrete, plain, reinforced, and prestressed. After summaries of previous experimental work, details of tests by Cowan and the author on torsion and, in particular, on combined bending and torsion of reinforced and prestressed concrete, are given. Formulas for the twisting moment, area of longitudinal and of lateral reinforcement, and a dual criterion of failure in combined bending and torsion are proposed. Conclusions from experiments on encased joists are stated. Methods of assessing the ultimate strength of concrete in torsion are suggested.

Direct models as an aid to reinforced concrete design

G. BROCK, *Engineering* (London), V. 187, No. 4857, Apr. 10, 1959, pp. 468-470

Reviewed by ABON L. MIRSKY

By "direct" models, author refers to models made of materials having stress-strain curves which are identical to those of the prototype except for a simple change of scale, for example, concrete or gypsum plaster reinforced with small diameter mild steel threaded rods, as distinguished from "indirect" models such as those used with the Beggs deformeter. Paper covers the advantages of model investigations with particular reference to direct model testing, the laws of structural model analysis, gypsum-plaster models, and the ultimate strength of reinforced plaster models. For the last, good agreement is shown between experimental results and theoretical values found by C. S. Whitney's analysis for concrete (*Transactions*, ASCE, V. 107, 1942, pp. 251 ff.)

While most of the material is not new, paper presents it in a form which will be of interest and value to structural engineers.

Moment redistribution in continuous beams reinforced with plain and deformed bars

K. HAJNAL-KONYI and H. E. LEWIS, *Proceedings of a Symposium on the Strength of Concrete Structures*, Cement and Concrete Association (London), 1958, pp. 158-200

AUTHORS' SUMMARY

Tests to failure have been made on 14 two-span continuous beams to compare the redistribution of bending moments with Tentor bars and with mild steel bars as tensile reinforcement. Six beams were over-reinforced and eight under-reinforced at the central support. In each group, half of the beams were in low-strength, and half in high-strength concrete. Two beams of each type were tested, with the exception of the beams over-reinforced at the central support with mild steel, of which only one was tested in each case.

Except for one case with shear failure, the ultimate loads on beams reinforced with Tentor bars were greater than those on beams with comparable mild steel reinforcement. Redistribution of bending moment took place in all cases. With beams in low-strength concrete over-reinforced at the central support, failure occurred by crushing of the concrete at the central support. With the same type of reinforcement in high-strength concrete, failure occurred at the support only after the yield point (in the case of Tentor bars the proof stress) had been exceeded in the span. In beams of high-strength concrete under-reinforced at the central support, in the case of mild steel bars the yield point was exceeded in the span when failure occurred at the central support by extensive elongation of the steel which caused crushing of the concrete, while, in the case of Tentor

bars, the proof stress was exceeded in the span when failure occurred at the central support by fracture of the bars. Beams with similar reinforcement in low-strength concrete failed in shear at the central support before the yield point or proof stress was reached.

Strength of concrete members under dynamic loading

S. C. C. Bate, *Proceedings of a Symposium on the Strength of Concrete Structures*, Cement and Concrete Association (London), 1958, pp. 487-525

Author's Summary

Presents a review of the available experimental data on the strength of reinforced concrete and prestressed concrete members under repeated loading and under impact or impulsive loading. It is concluded that the laboratory tests indicate that each form of construction has, in general, an adequate resistance to fatigue failure. Adverse corrosive conditions in service could, however, invalidate this conclusion.

Under impact loading, prestressed concrete members have a high capacity for recovery from severe deformation, while reinforced concrete members have a high capacity for energy absorption with little capacity for recovery. For both forms of construction, transverse reinforcement fulfills a more important role under impact loading than under static loading.

Strength of concrete walls under axial and eccentric loads

A. E. Seddon, *Proceedings of a Symposium on the Strength of Concrete Structures*, Cement and Concrete Association (London), 1958, pp. 445-481

Author's Summary

A review is given of information on the strength and behavior under axial and eccentric loads of monolithic concrete walls of types used in building construction. Dense concrete and no-fines concrete walls are considered, and their strengths under axial and eccentric loads distributed over the full wall length and concentrated over part of the wall length are indicated in so far as they have been investigated by laboratory tests. The effects on strength of the addition of reinforcement and of openings in the dense concrete wall are included.

Study of beam supports and column capitals in prefabricated structures (in Swedish)

Ake Holmberg, *Nordisk Betong* (Stockholm), V. 3, No. 4, 1959, pp. 349-354

Reviewed by Margaret Corbin

To be capable of rotation, the ends of precast concrete beams should rest on convex supports. The investigation dealing with column capitals comes to the conclusion that convexity, which is expressed by the ratio a/b, should be 0.05; that an asphalt felt insert should be provided between beam and column capital; and that reinforcement should be suitably distributed over the probable cross-section of rupture. A second investigation deals with brackets at the supports and presents a graphic analysis.

A note on the mechanism of diagonal cracking in reinforced concrete beams without shear reinforcement

R. Taylor, *Magazine of Concrete Research* (London), V. 11, No. 33, Nov. 1959, pp. 159-162

Author's Summary

An hypothesis is made concerning the mechanism of diagonal cracking in reinforced concrete beams without shear reinforcement; a test on a beam with pre-formed flexural cracks appears to support the hypothesis. The implications of such a mechanism of diagonal cracking are discussed.

General

Illustrations for publication and projection (ASA Y15.1-1959)

American Society of Mechanical Engineers and American Standards Association, 1959, 16 pp., $2

A guide to help authors prepare clear and legible illustrations for technical papers and articles. This pamphlet describes a standard method for preparing graphs, line drawings, and tabular illustrations for technical pub-

lications and slide projection. This ASA standard supercedes Z15.1 covering lantern slides and Z15.3 covering graphs for publications.

The standard emphasizes that simplicity, legibility, and unity are essentials of charts and graphs intended for use as visual aids. The standard presents practical guide rules and illustrates how they are used to achieve effective visual aids. An innovation is the functional integrated system provided for use with a standardized draft form. With this, the technical author can easily prepare effective designs while saving both production time and expense. Standard sizes for ordinary template lettering, symbols, and ruled lines are established for use with this system or with such variations of it as may be required for nonstandard drawings.

Certain large public works of the First Empire and their builders (Quelques grands travaux publics du premier empire et leurs maitres d'oeuvre)

A. Lomon, *Annales des Ponts et Chausées* (Paris), V. 129, No. 3, May-June 1959, pp. 345-363

Reviewed by Aron L. Mirsky

Three Director Generals of Bridges and Highways under Napoleon I are singled out: Cretet, Montalivet, and Molé, and public works projects conceived and executed in that era are discussed under the headings of highways, bridges, navigable waterways, and ports. The historically minded will find this engrossing.

Engineering college research review —1959

Engineering College Research Council, American Society for Engineering Education, University of Illinois, Urbana, Ill., 1959, 446 pp., $2

Lists over 10,000 research projects now underway in 118 engineering colleges in the United States. Expenditures on these research activities during the current year will total over $112,000,000. The review is intended to assist colleges, students, and government, industrial, and foundation sponsors of research by identifying the institutions conducting research in the areas of interest to each group. In addition to listing active research projects, information is also given about the research policies, research personnel, administrative officers, and magnitude of the research program for each of the member colleges and universities.

Concrete industries yearbook—1959

Pit and Quarry, Publications, Inc., Chicago, 21st Edition, 1959, 200 pp., $5

This edition is divided into three parts: concrete units, ready-mixed concrete, and prestressed concrete. At the end of each section is an index for reference to items in that section only. Likewise the advertising following each section primarily applies to the concrete application in that section. With this setup a producer interested in a particular production could save that complete section and discard the rest, still having all pertinent information.

After an introduction, Part 1 takes the reader through the entire procedure of production of concrete units. It commences with the methods of establishing a block business including plant layout. It then discusses the materials to be used, the manner in which the materials are used and the equipment used to handle the materials. It even goes into particular types of admixtures, automatic batching controls, compression testing equipment, etc. The following special topics are likewise covered: color, texture, precasting, wall reinforcement, and concrete pipe.

Part 2 deals similarly with ready-mixed concrete, commencing with the establishment of a business, plant layout, material analysis, and batching control. A special section deals with supervision and administration including office operations and record keeping.

Part 3 on prestressed concrete, after introductory information defining pre-

stressed concrete and explaining the advantages of prestressing, gives a history of the development of prestressing in other countries and in the United States. The article discusses the setup of a prestressing plant, materials to be controlled and used, and the fabrication methods and equipment which must be carefully utilized; special equipment and fittings are shown. Finally applications are discussed and pictured. Brief mention is made of design requirements and specifications as developed by various societies.

Bibliography on filing, classification, and indexing systems for engineering offices and libraries

ESL Bibliography No. 14, Engineering Societies Library, New York, 1960, 33 pp. (multilithed) $2

A selected annotated list of 155 references. The references are to books, pamphlets, and magazine articles on filing, classification and indexing, lists of subject headings, and hand-sorted punched-card systems suitable for organizing small collections of engineering books, notes, correspondence, abstracts, reprints, drawings, maps, and manufacturers' catalogs. Some of the listed items are broadly applicable; others are related to specific subject fields. There is a subject index as well as an introduction in which selection and use of the systems are discussed.

Directory of building research and development organizations

Conseil International du Batiment Pour la Recherche L'Etude et la Documentation, c/o Bouwcentrum, Rotterdam, 1959, 163 pp., 15 Swiss F

This directory, a revised edition of the one published by the United Nations in 1951, presents information on organizations engaged in the study of building problems. It includes European organizations, international organizations, and nonEuropean organizations which are members of the Conseil International du Batiment Pour la Recherche L'Etude et la Documentation (CIB) [International Council for Building Research Studies and Documentation]. The directory lists general and specialized organizations and presents in addition to the name and address of each organization, information on the staff, field of work, and publications.

Special services (welfare and recreation) facilities

Technical Publication, Navdocks TP-Pw-13, Bureau of Yards and Docks, Department of the Navy, Sept. 1953, 62 pp., $1.75 (Order Report PB 151980 from office of Technical Services, U. S. Department of Commerce)

The purpose of this publication is to establish Bureau of Yards and Docks principles and practices concerning standards, design, planning, construction, alteration, repair, upkeep, maintenance, and inspection of special services facilities at all naval shore activities.

It presents design factors, formulas, and graphs to be used as engineering guides in the solution of structural problems involving construction methods. It covers auditoriums, swimming pools, outdoor recreation facilities, gymnasiums, navy exchanges, chapels, and recreation buildings.

Index of publications of the Road Research Laboratory: 1933-1958

Road Research Laboratory (England), July 1959, 118 pp.

The work of the Road Research Laboratory in England has been described in many publications ranging from official reports issued by H. M. Stationery Office to papers contributed by staff members to society publications and the technical press. This index lists all publications from 1933 to the end of 1958 and includes brief summaries of the contents of *Technical Papers* and other Road Research Laboratory publications. Papers and articles by members of the laboratory staff are listed in a separate section. A subject index to all publications is included as well as directions for purchasing or, in the case of out-of-print issues, borrowing official publications.

ACI NEWS LETTER

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Record Turnout —

ACI Convention Reviews Progress and Looks to Unsolved Problems

A NEW ANNUAL CONVENTION registration record was set in New York during the 56th Annual ACI Convention, March 14-17, when more than 1000 registered for the 4-day event.

A full program of technical committee meetings, papers, and reports attracted leaders in concrete and related construction fields from all over the world. In reporting progress in the various fields of research, design, and construction techniques, the various sessions also gave hints of things to come — both the problems to be solved and some of the new developments that will probably lead to the solution of some of the vexing problems that still remain unsolved.

A day and a half, March 14-15, were devoted to technical committee meetings. The first general session on Tuesday afternoon, March 15, was devoted to ACI committee reports.

Concurrent sessions began on Wednesday, March 16, with model tests and highways being featured in the morning and materials and design and structural research being featured in the afternoon. Thursday morning continued the concurrent sessions with a symposium on restoration of deteriorated concrete and a session on design and construction. The annual research session on Thursday afternoon, March 17, spotlighted some of the concrete research in progress at institutions throughout the world.

Two ACI presidents get together with the chairman of the New York convention committee (left to right)—Phil M. Ferguson, retiring president, Roger H. Corbetta, New York, and Joe W. Kelly, newly elected president

An exhibit of technical products and various social events rounded out the New York program.

Technical committee meetings

More than 40 technical committees and subcommittees held working sessions during the convention. A day and a half was devoted exclusively to this important part of Institute work, and some groups met in the evenings and even prior to the official start of the annual meeting. There were also administrative meetings of the Board of Direction and the Technical Activities Committee, as well as a brief informal session of ACI chapter representatives.

All of the technical committee meetings were open to Institute members except in a few cases where committee work had progressed to the point that committees requested a closed or executive session.

Committees meeting were: Committee 115, Research, executive group; Committee 116, Nomenclature; Committee 201, Durability of Concrete in Service; Committee 207, Properties of Mass Concrete; Committee 208, Bond Stress; Committee 209, Creep and Volume Changes in Concrete; Committee 212, Admixtures.

Committee 213, Properties of Lightweight Aggregate Concrete, held a meeting of the complete committee and also scheduled four subcommittee meetings. Other committees were: Committee 214, Evaluation of Results of Strength Tests of Concrete; Committee 216, Fireproofing or Fire Protection of Structures.

Committee 318, Standard Building Code, held several lengthy executive sessions and Subcommittees No. 2, 8, and 15 of Committee 318 also met.

Other committees were: Committee 324, Precast Reinforced Concrete, Thin Sections; Committee 326 (ACI-ASCE), Shear and Diagonal Tension; Committee 328 (ACI-ASCE), Limit Design; Committee 331, Structures of Concrete Masonry Units; Committee 334, Concrete Shell Structures.

Committee 335, Deflection of Concrete Building Structures; Committee 336, Combined Footings; Committee 339, Allowable Stresses in Reinforcement; Committee 401, Specifications for Structural Concrete; Committee 402, Concrete Floor Finishes; Committee 505, Design and Construction of Reinforced Concrete Chimneys.

Committee 609, Consolidation of Concrete; Committee 611, Inspection of Concrete; Committee 612, Recommended Practice for Curing Concrete; Committee 613, Recommended Practice for Proportioning Concrete Mixes, and one of its subcommittees; Committee 621, Aggregates; Committee 622, Formwork for Concrete.

Committee 623, Cellular Concretes; Committee 712 (ACI-ASCE), Precast Structural Concrete Design and Construction; Committee 714, Concrete Bins and Silos; Committee 716, High Pressure Steam Curing; Committee 717, Practice in Low Pressure Steam Curing; and Committee 805, Application of Mortar by Pneumatic Pressure.

Concrete Progress and Problems

The first general session on Tuesday afternoon was aptly prefaced with the address of retiring ACI President Phil M. Ferguson, in which he discussed ACI's expanding role in concrete technology and some of the major problems facing the industry. The session was opened with a welcome to New York by Roger H. Corbetta, of Corbetta Construction Co., chairman of the New York convention committee.

President Ferguson pointed to the increased role that ACI is playing internationally, the increased activity of ACI technical committees, and some of the problems facing the industry. (The complete address can be found on p. 1097 of this month's JOURNAL.)

In speaking of industry problems, President Ferguson said, "One should not gather . . . that the work of ACI is complete nor that the concrete industry has reached the place where it can be content with past honors." As examples of the problems that are facing the industry, he singled out such subjects as uniformity of concrete strength and properties, adequate inspection of construction, and adequate research.

"The industry is well aware," he said, "that concrete strength, under properly controlled conditions, can be maintained reasonably uniform quite close to the desired level." However, he pointed out that in many of today's construction jobs variation in concrete strength on the job is one of the most costly factors now entering into concrete construction. He emphasized that the industry must come up with a better system of concrete mix control than it now uses.

Another major problem lies in the lack of proper engineering inspection, which is more a matter for the architects and engineers than it is for the contractor. President Ferguson called attention to the advantages to be gained from adequate supervision and inspection by the architects and engineers.

Major emphasis was placed on the lack of research in the field of concrete and reinforced concrete—in fact, in the construction industry as a whole. He said, "There are many questions crying

Harry C. Delzell (left) was presented with the Henry L. Kennedy Award by ACI President Ferguson in recognition of his outstanding contributions to the advancement of the objectives of the Institute

Vice-President Lewis H. Tuthill (right) "takes a break" from a session to admire the Construction Practice Award given to J. F. Camellerie

for answers and speaking generally there is all too little money with which to provide the answers." He continued, "I doubt that the construction industry averages 0.1 percent of its funds directed to research."

He suggested that ACI form a "Research Booster Club" to promote greater research in the concrete construction industry as well as to help raise funds to support such undertakings. He emphasized that research is a profitable investment, not an expense.

Three committee reports that then followed emphasized even further the need for more research by the concrete industry. Progress reports were presented by ACI-ASCE Committee 326, Shear and Diagonal Tension; by Committee 401, Specifications for Structural Concrete, and by Committee 609, Consolidation of Concrete.

Shear and Diagonal Tension—Progress report of ACI-ASCE Committee 326 presented by Eivind Hognestad, chairman.

In 1950 a joint committee of ACI and the American Society of Civil Engineers was formed with the assignment of developing methods for designing reinforced concrete members to resist shear and diagonal tension consistent with the new ultimate strength design methods. The committee immediately initiated a program of tests of beams without web reinforcement. Although this initial program did not fulfill the committee's assignment, it did focus attention on the complexity of shear and diagonal tension, on the possible lack of safety with present design procedures, and on the general lack of knowledge concerning the fundamental nature of shear and diagonal tension.

As a result, shear and diagonal tension became the major problem of reinforced concrete for this decade and a few structural failures added momentum to the intensive efforts devoted to its solution. During this 10-year period, a large proportion of the research work in reinforced concrete was assigned to shear and diagonal tension. Investigations and test programs were initiated and sponsored by many interested organizations. When combined, these projects rival the famous ACI column investigations of the 1930's, both in magnitude and in scope.

It is the purpose of the report being prepared by Committee 326 to consolidate the thoughts and knowledge gained from the various investigations and to formulate from them safe, workable design procedures.

In some areas of the problem, research has amassed sufficient data from which reliable *empirical design procedures* may be established taking into account the major variables affecting design strength. Other areas, such as the effects of torsion, have received little attention in the laboratory. Further research work should be encouraged not only to explore other areas of the problem but also to determine the rational behavior of shear and diagonal tension from which even better design procedures may be formulated.

The final report, which is still being worked on by the committee, will probably be divided into the following sections: historical review, including development of the classical working

stress design procedure, review of ACI and other codes, and review of structural failures; outline of research work since 1945 on combined stresses, beams and girders, frames, knee frames, culverts, and slabs and footings; principles of shear; beams without web reinforcement; beams with web reinforcement; members subjected to shear, bending, and axial load; slabs and footings; and members subjected to shear, bending, and torsion.

Specifications for Structural Concrete—Progress report of ACI Committee 401 presented by George H. Nelson, chairman.

The committee is preparing material to serve, by reference, as the section on structural concrete in building specifications. It is expected that by the proper use of alternates and by a few provisions in a project specification that the entire specification as being prepared by Committee 401 may be used by reference, thus obviating the necessity for writing an entire section on structural concrete in each job specification.

The committee work has been divided among 12 subcommittees, one for each major section of the specification. The first section will detail the scope of the specifications, list relevant specifications and recommended practices of ACI and other organizations, and list standard definitions. Other sections will cover materials; proportioning; mixing; forms; placing, curing, finishing, and protection; architectural concrete; reinforcement; construction joints; inspection and testing; an evaluation procedure to determine what is and what is not acceptable concrete; special concretes; and possibly a special section on shop drawings.

New York Registration Records

Converging from 40 states, the District of Columbia, Puerto Rico, Canada, Guatemala, Mexico, Norway, and Switzerland, 1010 registrants attended the ACI 56th annual convention. This also included 137 students of engineering colleges.

Following are registrations from the various states and countries:

New York	345	Washington	5
New Jersey	105	Minnesota	4
Illinois	89	Mississippi	4
Pennsylvania	65	North Carolina	4
Canada	61	Kansas	3
Michigan	37	West Virginia	3
Ohio	37	Arizona	2
California	36	Florida	2
District of Columbia	31	Kentucky	2
Colorado	26	Maine	2
Massachusetts	20	Mexico	2
Texas	13	Nebraska	2
Connecticut	12	Puerto Rico	2
Missouri	12	Guatemala	1
Georgia	11	Hawaii	1
Maryland	9	Louisiana	1
Wisconsin	9	North Dakota	1
Indiana	8	Norway	1
Virginia	8	Oklahoma	1
Iowa	7	Rhode Island	1
Alabama	6	Switzerland	1
Tennessee	6	Utah	1
Delaware	5	Wyoming	1
South Carolina	5		

Consolidation of Concrete — Report of ACI Committee 609 presented by Joseph J. Waddell, chairman.

This report was published in the April, 1960, JOURNAL, p. 985.

The first chapter reviews briefly the history of consolidation of concrete. This is followed by a discussion on advantages of vibration, types of equipment and operating frequencies, forms, and concrete mixes (structural, mass, pavement, floor, lightweight, heavyweight).

The second chapter is devoted to recommended practices and gives information on amount of vibration, effect on entrained air, over-vibration, revibration, and use of retarders. Recommended techniques are outlined for structural concrete, slabs, mass concrete, pavement (thick, thin, and reinforced), earth-moist concrete, lightweight concrete, heavyweight concrete, concrete block, and precast products.

Model Tests–An Aid in Research and Design

The actual behavior of structures is most easily studied in laboratories, but space requirements there preclude tests on full scale prototypes. Hence the structural model plays an important role in fundamental research and also in applied design studies.

One of the concurrent sessions on Wednesday morning was devoted to model tests with papers discussing bridge models, dam models, and thin-shell roof models.

Prof. Joe W. Kelly, newly-elected ACI president, Department of Civil Engineering, University of California, Berkeley, was session chairman.

Tests of a Rigid-Frame Bridge Model to Ultimate Load — Ching-Sheng Wu, research engineer, California Institute of Technology, Pasadena; Arpad A. Pap, assistant professor, and D. H. Pletta, professor and head, Department of Engineering Mechanics, Virginia Polytechnic Institute, Blacksburg.

This paper was presented as part of the work of ACI Committee 314, Rigid Frames for Buildings and Bridges.

A $\frac{1}{10}$ scale model of an existing skewed reinforced concrete rigid-frame bridge was constructed and tested both within the elastic range and up to initial cracking in the concrete and thence to ultimate load. The model was fabricated of reinforced concrete using $\frac{1}{8}$-in. deformed reinforcing. A total of 57 SR-4 rosette gages was used to measure strains on the surface of the concrete at selected positions. An additional 108 SR-4 gages were fastened to the steel reinforcing. Deflections were measured at 11 points of the deck, and horizontal reactions determined by suitable dynamometers.

Part of the strain data was converted to unit stress for several loadings. Concentrated loadings located at or near the center of the deck produced large local strains, especially in tension. The model was essentially elastic up to about $\frac{1}{3}$ ultimate load and failed in the abutment at the knee.

Structural Models Evaluate Behavior of Concrete Dams — Jerome M, Raphael, associate professor of civil engineering, University of California, Berkeley.

An unusual arch and buttress shape dam was proposed as one possible design for the 750 ft high by 5000 ft long Oroville Dam. A plaster-celite, 1:200 scale model was cast in Fiberglas molds to duplicate the central portion of the dam. Live load tests were performed on the dam, simulating the uniformly increasing water pressure by a series of rubber bags under stepped pneumatic

pressure. The control apparatus was described, and the results of the water load tests given. A new method of dead load testing was devised by which the construction stresses as well as the dead load stresses of the completed dam could be determined. Dead load stresses were shown for various stages of construction, as well as the stresses under combined live and dead loads. The technique for dead load testing offers opportunity for checking the stresses in concrete dams under various proposed construction programs and should be especially useful for controlling the construction of the overhanging portions of doubly-curved arch dams.

Glen Canyon Dam Structural Model Tests—George C. Rouse, engineer, U.S. Bureau of Reclamation, Denver, Colo.

Structural model tests were made of Glen Canyon Dam, a 700 ft high arch structure which is presently being constructed by the Bureau of Reclamation as part of the Colorado River storage project.

The model was constructed of plaster-celite to a 1:240 scale. To satisfy a condition assumed in the structural analysis of the dam by the trial load method, the modulus of elasticity of the material from which the foundation and abutments were formed is 1/6 of the modulus of elasticity of the material used in the model dam. Model construction methods, loading, and testing technique, were discussed.

Experiments with Thin-Shell Structural Models—Joseph L. Waling, professor of structural engineering, Purdue University, Lafayette, Ind., and Longin B. Greszczuk, Douglas Aircraft Co., Culver City, Calif.

A new method of forming hyperbolic paraboloidal concrete shell structures was presented. The method which depends upon the special geometric characteristics of the hyperbolic paraboloid makes use of wire-supported Styrofoam with its high strength to weight ratio, as a temporary shell material to support the permanent shell material during curing. The Styrofoam thus serves temporarily as a structural material in lieu of extensive formwork and remains as insulation and vapor barrier throughout the life of the structure.

Experiments which led to the proposed construction procedure made use of space models, table-sized construction models, and a laboratory construction model. Results of the model studies indicate possibilities of success with full-scale constructions using the proposed procedure.

Among those attending the 56th annual ACI convention in New York were eight civil engineering professors from India. These men are all pursuing master of science degrees at the University of Colorado under a program sponsored by the International Cooperation Administration.

One of their missions in the United States is to learn American methods of engineering education and engineering design and construction. Additionally, they are to improve their technical competence and may pursue a higher academic degree in the process. Their transportation to and from this country plus their tuition, books, and fees are paid by the program as well as subsistence allowance during their stay. Funds are also provided for attendance at technical society conventions such as the ACI meeting and the annual ASCE meeting which the Colorado group also attended.

The professors studying at the University of Colorado and their home institutions are as follows:

Shreeram K. Damle	University of Baroda
Pasala Dayaratnam	University College, Andhra University
V. Dwarakaprasad	University College of Engineering, Bangalore
Natarajan Krishnamurthy	National Institute of Engineering, Mysore
Anantanarayanan K. Narasimhan	Coimbatore Institute of Technology
Edhala S. Reddi	Unattached
Bibhuti P. Sinha	Muzaffarpur Institute of Technology
Kandasamy Srinivasan	Annamalai University

Similar groups are studying at Cornell University, University of Missouri, University of Wisconsin, and University of Illinois.

POZZOLITH makes good concrete better

PONTECORVO CANAL—Owner: Electric Company of Rome • Designing Engineer: Ing. Silvano Zorzi
Contractor: Farsura, both of Milan, Italy.

Flexible concrete plates drape-shaped to line canal bed

New technique uses prestressed POZZOLITH concrete

The Pontecorvo Canal carries water five miles to this hydroelectric project owned by the Electric Co. of Rome, Italy. Because the canal crosses irregular terrain, running partly in fill and partly in cut, a flexible concrete lining able to adjust itself was specified.

Inch-thick plates of prestressed concrete, four feet wide and up to 98 feet long, were used to line the paraboloid canal bed. A gantry crane gripped both ends of each precast plate, causing it to drape into a catenary curve when lifted. Placement and alignment of successive plates in the prepared mortar bed over the masonry base thus required a minimum of skilled labor. The joints were then sealed with non-shrink EMBECO® mortar.

Concrete for this project had to be durable, impervious and economical. To help assure this performance POZZOLITH was specified, because Ing. Silvano Zorzi's previous experience with it showed that POZZOLITH makes good concrete better. Here, POZZOLITH in the mix reduced the total water content and improved all of concrete's desirable qualities, giving less shrinkage, lower permeability and higher strength.

Your local Master Builders field man will be glad to discuss your concrete requirements with you, and show that POZZOLITH concrete is higher in quality and more economical than plain concrete, or concrete made with any other admixture.

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*POZZOLITH *is a registered trademark of The Master Builders Co. for its concrete admixture to reduce water and control entrainment of air and rate of hardening.*

Highway Progress Reviewed

Highways received attention at the other concurrent session on Wednesday morning. One paper summarized the status of air-entrained concrete, another reported on the continuous placement of concrete on a three-span expressway bridge. The AASHO Road Test was described and the last paper was devoted to materials with a description of gravel beneficiation in Michigan. Session chairman was ACI Vice-President Lewis H. Tuthill, of the California State Department of Water Resources, Sacramento.

Air-Entrained Concrete After 20 Years —E. A. Finney and M. G. Brown, respectively, director and chemical research engineer, Research Laboratory Division, Office of Testing and Research, Michigan State Highway Department, East Lansing (Presented by Ray Durfee).

A summary survey of air entrainment related primarily to highway construction, the survey dated back to the late 1920's, briefly describing the principle of air entrainment and telling how the principle was developed.

Many of the early experimental test pavements (1938-1942) were described together with results from recent field observations. Michigan's experiences with air entrainment and the Long-Time Study of Cement Performance conducted by the Portland Cement Association were given special consideration.

These researches have established beyond any possible doubt the benefit of air entrainment in improving the resistance of concrete pavement to scaling. Further, they have conclusively demonstrated that this improvement in resistance to scaling due to the purposely entrained air overshadowed all other variables included in the respective tests, when the amount and character of the entrained air was in accord with established practice.

Although air entrainment has definitely proved its value in producing frost and salt resistant concrete, problems have been introduced which have not been satisfactorily solved. Mix proportioning and field control still present serious difficulties because the optimum air content lies within a relatively narrow range and the amount of air introduced into the mix depends upon many different factors.

Evidence also supports the use of air-entrained concrete in bridge structures. In the case of bridges, proper precautions must be exercised in the design and control of the concrete mixture to insure satisfactory bond and strength. Further, bridge structures because of their piecemeal construction require the utmost in inspection and field control concerning the placing, finishing, and curing of the concrete to insure scale-free surfaces in all areas of the structure. Prestressed air-entrained concrete members are now being successfully fabricated in large numbers for highway bridge construction.

The use of air-entrained ready-mixed concrete in highway pavements and structures presents additional problems in field control and inspection which require immediate attention on the part of both producer and consumer.

The advent and application of modern automatic batching and mixing equipment, as well as the development of improved equipment for dispensing air-entraining agents and measuring air content in the fresh concrete has done much to insure the manufacturer of quality air-entrained concrete and to encourage its use.

Continuous Integral Deck Construction—Placing Structural Deck on Three-Span Continuous Bridge Units—H. B. Britton, senior structural specifications writer, State of New York Department of Public Works, Albany.

Experience in New York with retarding densifiers indicated feasibility of the continuous placement of concrete on three-span continuous structures.

A program was therefore set up for the deck of the three-span continuous units of the Bruckner Expressway as follows: (1) Theoretical deformation calculations were submitted prior to placing concrete. (2) Deformation measurements were taken continuously during the entire placement operation. (3) Final deformation readings were taken at the conclusion of the placement of all concrete. (4) A method was provided to insure the proper location of the reinforcing steel, together with the required cover. (5) The addition of the retarding densifier was so controlled that the plasticity of the concrete for the final placement was relatively the same as that placed at the first placement at the time of placement of final concrete. (6) A complete physical record was kept for the concrete placed for this operation, including gradation of the aggregates together with temperature and moisture content. The report also included slump, air content, water reduction factor, temperature of the concrete when placed, ambient temperature and humidity, weather and other climatic conditions, timing of progressive placement, time when burlap drag was accomplished, time when curing was begun, method of curing, and results obtained.

The AASHO Road Test—A Background Report—F. H. Scrivner, rigid pavement research engineer, AASHO Road Test, Ottawa, Ill. (Presented by W. R. Hudson).

The AASHO Road Test is a $25 million project designed as a study of the performance of highway pavements and short-span bridges of certain structural designs when subjected to controlled traffic with certain specific loadings. The paper described the basic concepts of the project, the physical layout of the test pavements and bridges, the various measurement programs and associated instrumentation, and the methods being used to analyze the data.

Looking Ahead to

1961 in St. Louis

ACI's Technical Activities Committee is already shaping up program plans for the 1961 convention to be held in St. Louis, February 20-23.

Those who intend to offer papers for presentation at the 57th annual meeting should write to Institute headquarters before July 1, furnishing a synopsis which should make clear the scope of the proposed paper and indicate features that the author thinks will justify its inclusion in the program. Contributors should have preliminary drafts of manuscripts in the hands of the Technical Activities Committee for appraisal and acceptance by Sept. 15, 1960, and final manuscripts of accepted papers will be due Jan. 1, 1961.

From the replies received and suggestions from other sources, TAC will select papers to make up each convention session.

Write Institute headquarters now about the papers you want to see on the 1961 program

The 368 test sections provided for rigid pavement research can be divided into five broad categories according to the principal experiments being performed on the sections: (1) main experiment, (2) paved shoulder experiment, (3) special strain, temperature, and curling experiments, (4) subsurface studies, and (5) weather effects study.

It was mentioned that the principal objective of the main experiment will have been largely accomplished when the data now being gathered are used to derive a group of equations of the following general form:

> The *performance* of a pavement = a known function of the *pavement design*, the *applied load*, and the *number of applications* of the applied load.

It is planned that all of the results from the test road will be published by the Highway Research Board in the near future.

There were serious discussions too — Prof. Charles H. Scholer (left) and Richard C. Mielenz

Gravel Beneficiation in Michigan—F. E. Legg, Jr., associate professor of construction materials, University of Michigan, Ann Arbor, and W. W. McLaughlin, testing and research engineer, Michigan State Highway Department, Lansing.

The gravel industry in southern Michigan has been particularly active in recent years in applying special processes to remove deleterious particles from the glacial gravels which abound in the region.

Best success has been achieved using heavy media separation, but presence of high density deleterious particles in at least one deposit indicates limitations of this process when both high and low specific gravity deleterious particles are in the same gravel.

Experience so far with the jig method has proven less successful since it does not appear to make as close discrimination as to specific gravity as these gravels demand. For example, in the case of heavy media processing, it has been observed that substantial changes in gravel quality may result, for some of these deposits, by making a media change of only 0.05 specific gravity.

The cage mill and elastic fractionation, or "bounce" method, appear to not give sufficient consideration to the fact that, for some of these gravels, the deleterious particles may be quite hard and resilient and thus not be removed by either of these processes. This is particularly the case with chert.

Probably the most important lesson to be learned from Michigan's experience with these beneficiation processes is that the particular method chosen must be carefully matched to the needs of each deposit. Different gravels may demand entirely different processes of beneficiation. The most satisfactory means of determining the technical feasibility—and this has been done on numerous occasions—is to run a truckload or two of the gravel to be investigated through an existing beneficiation plant. Generous size lots of the gravel are then provided for whatever tests may be desired.

The gravel producer must, of course, consider the economics of the installation. The original investment will be substantial and he certainly should con-

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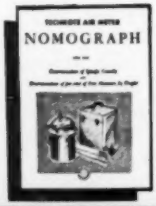

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sider market potential, means of disposal of waste product, and such matters.

Beneficiation has enabled some gravel producers not only to survive competition, but to be able to furnish premium aggregates complying with rigid specifications for architectural concrete where severe exposure conditions are anticipated.

Materials Session World-Wide in Scope

One of the concurrent sessions on Wednesday afternoon was devoted to materials, with two of the reports coming from South Africa. Subjects included: effect of incomplete consolidation on various properties of concrete, freeze-thaw resistance of lightweight aggregate concrete, effect of maximum size of coarse aggregate on concrete properties, and high-magnesia portland slag cement concretes. Session chairman was ACI Past President Frank Kerekes, dean of the faculty, Michigan College of Mining and Technology, Houghton.

Effects of Incomplete Consolidation on Compressive and Flexural Strength, Ultrasonic Pulse Velocity, and Dynamic Modulus of Elasticity of Concrete—M. F. Kaplan, head, Civil Engineering Division, National Building Research Institute, South African Council for Scientific and Industrial Research, Pretoria.

Concrete mixes having cement:aggregate:water ratios of 1:3:0.35, 1:6:0.50, and 1:9:0.66, by weight, were used to make 20 x 4 x 4-in. beams and 6-in. cubes for resonant frequency and flexural and compressive strength tests. The tests were done after 7, 28, and 91 days. Pulse velocity measurements were also made on all specimens before carrying out the other tests. The amount of consolidation was varied by vibrating the concrete for different lengths of time. The maximum void content was 32 percent.

The effects of incomplete consolidation on compressive and flexural strength, pulse velocity, and dynamic *E* were reported and the interrelationships between these properties examined.

(The complete paper appeared in the March, 1960, JOURNAL.

Freezing and Thawing Tests of Lightweight Aggregate Concrete—Paul Klieger, senior research engineer, Applied Research Section, and J. A. Hanson, senior development engineer, Products and Applications Section, Research and Development Association, Skokie, Ill.

Nine lightweight aggregates and one natural sand and gravel aggregate were used in concrete subjected to laboratory freezing and thawing tests and, in some cases, to tests for resistance to deicer scaling. Concretes were prepared at two levels of compressive strength: 3000 psi and 4500 psi at 28 days. Both non-

Take our word for it — not all of the convention talks were theoretical. Roger H. Corbetta (left), chairman of the New York convention committee, gets a chuckle out of President Joe W. Kelly's remarks

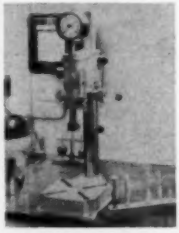
Darex Diary

by Mel Prior
Member, American Concrete Institute
Technical Service Manager, Construction Products
W. R. GRACE & CO.
Dewey and Almy Chemical Division
(No. 18 of a series)

ADMIXTURE CONTROL

THE RECENT ASTM Symposium on water-reducing and set-retarding admixtures underscored the growing recognition these products are gaining in the concrete industry — particularly those admixtures that are manufactured to exacting chemical specifications under controlled conditions.

Two such materials that have been developed by chemical research are WRDA, water-reducing admixture and DARATARD, a water-reducing and initial set-retarding admixture. Both of these admixtures are manufactured from raw materials purchased under rigid specifications. Control is strictly maintained throughout the manufacturing process. Then the final product is subjected to still further chemical control as well as actual tests to determine the uniformity of performance.



Tests such as the Proctor Needle Penetration Resistance (ASTM Designation C403-57T) are regularly employed to evaluate the ability of DARATARD to retard the initial set of concrete.

(Courtesy Bureau Public Roads)

To obtain best results, no admixture should be used without the benefit of sound technical advice. That's why we have trained our distributors and their sales personnel in the proper use of our products. These men are available for consultation and service in practically all parts of the world. They have had long experience with these materials and can help you gain maximum benefits when you use our admixtures.

Admixtures, like any other ingredient of concrete, must be properly proportioned and added at a controlled rate. Lack of adequate control often results in unnecessary waste of materials and produces less than optimum results. Batch plant and paver mix manufacturers recognize the need for adequate dispensing equipment and now furnish such equipment with many of their units. For our part, we have cooperated with these equipment manufacturers in the design and development of dispensers for our admixtures. What's more, our trained personnel are available for installing and calibrating this equipment.

If *you* have any problems on the use of admixtures and concrete, let our experts assist you. Simply write to:

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air-entrained and air-entrained concretes were prepared, using the aggregates in an air-dried condition and in a 24-hr saturated condition.

The results of these tests indicate the necessity for providing intentionally entrained air to attain a high level of durability, the importance of the moisture content of the aggregate, and the influence of strength level, i.e., water-cement ratio, on the durability. The results point to the desirability of evaluating a lightweight aggregate by means of laboratory freezing and thawing tests of air-entrained concrete made with the aggregate, as is generally done for normal weight aggregate.

Effects of Aggregate Size on Properties of Concrete—Stanton Walker, director of engineering, and Delmar L. Bloem, associate director of engineering, National Ready Mixed Concrete Association and National Sand and Gravel Association, Washington, D. C.

The report described research supplementing earlier work which indicated a lack of improvement in concrete strength resulting from increased maximum size of aggregate. The more recent program provided a broader range of classes of concrete and test variables for the purpose of checking the degree of applicability of the earlier finding.

The tests were made with maximum sizes of coarse aggregate ranging from 3/8 to 2½ in., using three cement factors, from 4 to 8 sacks per cu yd, both with and without air entrainment. Compressive and flexural strength tests were made at ages of 7, 28, and 91 days. Tensile-splitting tests were made at 28 and 91 days. Drying shrinkage measurements were also made on all concretes.

The tests showed that size of coarse aggregate exerts an influence on concrete strength independently of the water-cement ratio. Results indicated that increasing the maximum size of coarse aggregate may not necessarily be beneficial to concrete strength. Drying shrinkage was not substantially increased by reduced size of aggregate down to about 3/4 in.

Two ACI members were elected to honorary membership — William Lerch (left) and Charles H. Scholer

Properties and Uses of High-Magnesia Portland Blast-Furnace Slag Cement Concretes—N. Stutterheim, vice-president, South African Council for Scientific and Industrial Research, Pretoria (Presented by M. F. Kaplan).

Some of the properties of portland blast-furnace slag cements made from slags having from 13 to 20 percent magnesia were described, in particular those composed of 50 percent slag and 50 percent portland cement clinker plus gypsum. Two ways of blending these constituents were dealt with.

Blast furnace slag for this study came from three different mills each of which uses dolomite as fluxing material in its blast furnaces. The composition and properties of the slags and of cements made from them were discussed; the effects of dry and wet grinding of slag, of fineness of grind, and of portland cement content were determined experimetally. Autoclave expansion results were invariably low.

The performance of concrete made with these various types of cements was given in respect to workability, shrinkage, dynamic modulus of elasticity, Poisson's ratio, and compressive

and flexural strengths. The results showed that the slags have good cementitious properties.

Some examples were shown of full-scale constructional applications of portland blast furnace slag cements made from high-magnesia slags.

(The complete paper appeared in the April, 1960, JOURNAL.)

Design and Structural Research

Design and structural research were featured in the other concurrent session on Wednesday afternoon. Subjects included: high strength reinforcement, shrinkage and creep, effect of draped reinforcement on prestressed concrete beams, and ultimate strength design. ACI Past President Douglas McHenry, director of development, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., was session chairman.

High Strength Reinforcement in Concrete Structures—First Progress Report—ACI Committee 339, Allowable Stresses in Reinforcement, Robert Sailer, chairman.

A brief review by the committee of papers and research dealing with high tensile strength reinforcement in concrete showed that successful application of high strength steel may depend on factors other than stresses in steel alone. High stresses in steel will increase the deflections of beams, will increase the width of tensile cracks in the concrete, and the increased width of crack may affect the durability of structures. It is possible that these factors may be a deciding factor when designing structures with high tensile strength steels.

With the above considerations in mind various members of the committee have begun to study the available data in detail. As might be expected, some of the points considered could be abstracted readily into reports while others are much too involved and need considerable more study. That is particularly true in regard to predicting crack width and effects of crack width on corrosion. Climatic condition and exposure to it appears to have a major influence.

The committee reported briefly on four items: (1) historical background, (2) high tensile strength steel, (3) design methods, and (4) deflection.

Shrinkage and Creep of Concrete—Inge Lyse, professor of reinforced concrete and director of the concrete laboratory, Norway's Institute of Technology, Trondheim.

In the design of prestressed concrete structures it is necessary to take into account the effects of shrinkage and creep, which cause a reduction of the initial prestressing forces. This paper reported on tests of the four major factors that contribute to shrinkage and creep. Equations were derived, based on the data obtained from the tests, for determining the combined shortening effects of creep and shrinkage.

(The paper was published in the February, 1960, JOURNAL.)

Effect of Draped Reinforcement on Behavior of Prestressed Concrete Beams—James G. MacGregor, research associate in civil engineering; Mete A. Sozen, associate professor of civil engineering; and Chester P. Siess, professor of civil engineering, University of Illinois, Urbana.

Tests on 19 simply-supported prestressed concrete beams with draped reinforcement were described and compared with the results of tests on similar beams with straight reinforcement. The principal variables included: con-

crete strength, steel percentage, length of shear span, and the angle and type of drape profile. Web reinforcement was used in only five beams.

In the practical ranges of the variables, draping the reinforcement did not increase the "shear" strength of the beams, which for beams without web reinforcement corresponds to the inclined tension cracking load. A comparison of the behavior of beams with draped and straight wires showed that the detrimental effect of the drape on shear strength could be ascribed to the earlier formation of flexural cracks in regions of combined bending and shear and the consequent earlier development of inclined cracks. For extreme combinations of the critical variables, an inclined crack occurred prior to the formation of flexural cracks in its vicinity, in which case draping the wires caused an increase in strength which could be estimated on the basis of an uncracked section analysis.

The beams with draped reinforcement required more web reinforcement to produce a flexural failure than similar beams with straight reinforcement.

Ultimate Strength Design—Alfred L. Parme, principal engineer, Advanced Engineering Group, Portland Cement Association, Chicago, Ill.

The talk was divided into four broad categories: (1) Effect of shape of stress block on ultimate capacity—This section dealt with the difference between the rectangular, trapezoidal, and parabolic stress block and their importance from a practical point of view; (2) Factor of safety with respect to differences between design strength and cylinder strength for various percentages of reinforcement; (3) Ductibility of group sections with various percentages of reinforcement — In this section the amount of rotation available after yielding of steel was plotted for various percentages of reinforcement; and (4) Widths of cracks at various levels of steel stress at design loads.

Restoring Deteriorated Concrete

Highlighting the final round of technical sessions, five concrete experts described successful methods used for restoring deteriorated concrete at the symposium sponsored by ACI Committee 201, Durability of Concrete in Service. The session was presided over by the Committee chairman, Hubert Woods, director of research, Research and Development Laboratories, Portland Cement Association, Skokie, Ill. The papers covered conventional methods of repair, repair of concrete pavements, the prepakt concrete method, use of epoxy adhesives, and pneumatically applied mortar.

Conventional Methods of Repairing Concrete—Lewis H. Tuthill, concrete engineer, California Department of Water Resources, Sacramento.

Conventional techniques of repairing concrete were outlined. It was emphasized that a good concrete repair or restoration must be thoroughly bonded to the older concrete; must be sufficiently impermeable to prevent moisture reaching underlying older concrete; must, after drying, be free of shrinkage cracks through which water could reach the supporting concrete; must be resistant to freezing where this is a factor in weathering; and should match surrounding concrete surfaces.

On new or restoration work, all unsound concrete should be removed completely. Mr. Tuthill recommended shaping the excavations so as to make the

As retiring ACI President, Prof. Phil. M. Ferguson (left) is decorated for "valor" by Miss Rosette Tarricone, Corbetta Construction Co., and member of the local ACI committee

replaced material as secure as possible, and that surfaces be cleaned and moistened for maximum bond. For cleaning, sand blasting should be preferred to scrubbing with a wire brush.

Although moistening of older concrete is usually advised before bonding, Mr. Tuthill observed that on joint surfaces possibly more permanent bond can be secured when these surfaces are dry at the time they are overlaid with fresh concrete. The effort formerly expended on moistening before the repair is transferred to more water curing afterward.

He also advised that for the restoration of weathered spillways, piers, and other hydraulic structures, reinforced air-entrained concrete is probably preferable to shotcrete. Other factors affecting bond, permeability, cracking, durability, and appearance of concrete repairs were also detailed.

Repair of Concrete Pavements—Earl J. Felt, manager, Transportation Development Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

The patching of distressed areas of concrete pavements with bonded concrete was discussed.

Mr. Felt cited several instances where long-lasting bond of relatively thin layers of concrete (1-3 in.) to old base concrete was secured. Some of these streets are still in service 40 years after resurfacing.

Field investigations and laboratory studies conducted by the Portland Cement Association have uncovered basic requirements for good bond between new and old concrete. Most important is a clean, sound, old concrete surface. One group of tests has shown that cleaning and slight roughening of an old sound surface with hydrochloric acid developed acceptable bond in the range of 300 to 500 psi. Chipping the old surface with an electric chisel improved bond, but the most effective tool for mechanical cleaning and for removing unsound concrete was the Tennant machine. Sand blasting was not particularly effective in these tests.

Probably the most common defect in concrete pavements is spalling at the joints, frequently caused by poor jointing design or improper construction. If air-entrained concrete has been used, the spalls are hard, durable concrete and the shearing surface makes a good base for a bonded concrete patch. Many pavements made of non-air-entrained concrete still exist, however, and spalling at joints is also associated with concrete disintegration. Here, the base concrete may not be in good condition and considerable preparation will be required before patching with bonded concrete.

In addition to good surface condition of the old concrete, high quality grout and concrete, first class workmanship in placement and compaction, and proper jointing and curing are essential. Suggested techniques covering these procedures were detailed.

Prepakt Method of Concrete Repair—Raymond E. Davis, consulting engineer (director emeritus, Engineering Materials Laboratory, University of California), Berkeley.

Prepakt was first used in 1938 in the strengthening and stabilizing of the concrete lining of Muir tunnel on the Santa Fe Railroad near Pittsburgh, Calif. Since then, the prepakt method has found many applications in the restoration of old concrete and masonry structures. It has also gained favor for certain types of new construction where the proper placement of conventional concrete is difficult or impossible, such as underwater work.

Prepakt is essentially a concrete of gap grading which is produced by first placing the coarse aggregate in the forms, and then pumping a sanded grout, sometimes referred to as intrusion mortar or prepakt grout, into the voids of the aggregate mass. In the repairs of many old stone masonry and concrete structures by the prepakt method, neat cement grouting for the purpose of improving structural stability and reducing leakage is also carried out.

Mr. Davis pointed out that bond strength between new prepakt concrete and old conventional concrete is much higher than that obtained between new and old conventional concrete. This is probably due to the grout being applied under pressure. Prepakt's wetting properties and its fluidity is much greater than that of the usual mortar constituent of ordinary concrete.

Another property in which prepakt excels is low drying shrinkage, which is likely to be only about 1/6 as great as the usual mortar applied pneumatically and less than half as great as that of conventional concrete. This low drying shrinkage, thought to be due to point-to-point contact of the pieces of coarse aggregate, also promotes greater crack-resistance, Mr. Davis explained.

Also, based on laboratory tests and performance of field structures over a period of years, prepakt concretes of suitable materials and good workmanship resist weathering action caused by freezing and thawing better than conventional concretes. Later-age strength of prepakt concrete is also considerably greater.

K. S. Pister (left) and Boris Bresler, University of California, received the Leonard C. Wason Medal for noteworthy research for the work reported in their paper "Strength of Concrete Under Combined Stresses"

Mr. Davis described the composition of prepakt, mix proportioning, and also methods of test for fresh prepakt grouts and hardened prepakt concrete. He also specified the types of repairs for which the method is particularly well adapted.

Repair of Damaged Concrete with Epoxy Resins—Bailey Tremper, supervising materials and research engineer, California Division of Highways, Sacramento.

How the California Division of Highways uses adhesives and binders containing epoxy resins to repair concrete was outlined. Only epoxy resins of low viscosity find application as concrete repair materials. The bond strength of properly formulated adhesives is greater than that of concrete, as has been demonstrated in many tests. Rupture always occurs in the concrete, not in the adhesive bond, Mr. Tremper reported.

Adhesives and binders for highway use are formulated of epoxy resin, a plasticizer, and a curing agent. Depending on the viscosity at the time of mixing, a certain amount of finely

ground inert filler such as talc or silica can also be added.

For the repair of highways and bridges under traffic, quick hardening is desirable but this complicates the mixing and handling of the compound since rapid hardening formulations also have a short pot life. In general, small batches must be mixed and applied promptly. The choice of curing agent (generally organic amines) and the percentage used determines the pot life and the setting time.

In patching spalled areas with epoxy mortar or concrete, the binder used by the California Division of Highways contains no filler and has a pot life of about 20 min at 70 F. Under favorable conditions, the repair has been opened to traffic within 3 to 5 hr. In cool weather, metal channels are placed over the repaired area and they are warmed with torches.

Epoxy concrete is considered economical to use in fairly large volume on heavily traveled structures such as the San Francisco-Oakland Bay Bridge. But if substantial repairs are involved and if traffic conditions permit, it is more economical to use portland cement concrete bound to the original work with a coating of epoxy adhesive.

As in other concrete repair work, the surface to which an epoxy adhesive is applied must be scrupulously cleaned, preferably by sandblasting. Concrete may be damp but must be free from surface moisture when the adhesive is applied.

Mr. Tremper concluded his paper by discussing possible variations in formulation for specific uses, methods of application to obtain strong and durable repairs, and a typical formulation for general use.

Experience with Pneumatically Placed Mortar for Restoring Concrete Structures—O. N. Kulberg, chief construction engineer, Southern California Edison Co., Los Angeles.

The experiences of the Southern California Edison Co. during the past 40 years in restoring concrete structures with pneumatically placed mortar (shotcrete) was described.

In addition to use for ordinary concrete repair, several million square feet of pneumatically placed mortar are employed in the company's complex of canals, flow lines, and dams. In no instances, Mr. Kulberg explained, has this mortar undergone direct deterioration from weathering such as the scaling and spalling shown by low air-entrained conventional concrete under severe exposure.

However, there are areas of severe weathering in the Sierra Nevada Mountains where difficulty has been encountered in pneumatically placed mortar restoration work on several dams. Failure of the parent material upon which the mortar had been placed, or lack of understanding on how to maintain it

CELLULAR CONCRETES

by Rudolph C. Valore

Price $1.00

Reviews methods of preparation and physical properties of moist and high-pressure steam-cured cellular concretes, ranging in density from 10 to 100 lb per cu ft, as they have evolved in Europe in recent years.

Data are given for Swedish, Danish, Russian, German, British, and Belgian materials, and for experimental mixtures prepared at the National Bureau of Standards. 44 pages.

Order from Publications Department, American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich.

were the chief points of difficulty. Generally, though, all pneumatically placed mortar regardless of how maintained has had sufficient longevity to justify its use, Mr. Kulberg said.

Edison Co. experience points up the fact, Mr. Kulberg said, that reinforced and dowelled pneumatically placed mortar, when properly maintained, is the most economical method of restoring disintegrating concrete where restoration is relatively shallow and over large irregular areas. Regular inspections for the sake of preventive maintenance are all important.

Design and Construction

The other concurrent session on Thursday morning was devoted to design and construction — covering such subjects as: welding of reinforcing steel between precast units, effect of floor concrete strength on column strength, shear strength of reinforced concrete beams, and design and construction of New York's Pier 40. Session chairman was Raymond C. Reese, consulting engineer, Toledo, Ohio, and newly elected ACI vice-president.

Welding of Reinforcing Steel Between Precast Concrete Units — J. Neils Thompson, Hudson Matlock, and A. A. Toprac, respectively, director of Balcones Research Center and professor of civil engineering, and associate professors of civil engineering, University of Texas, Austin.

A previous investigation served to establish that the damage to welded splices between unrestrained precast concrete units consisted primarily of cracking due to differential thermal expansions. The present study was intended to establish the effects of the controlling variables and to evaluate the damage.

Specimens consisted of pairs of concrete blocks cast with a deformed reinforcing bar projecting from the end of each block. The bars were connected with a 60-deg V-butt weld, performed at a reasonably rapid rate.

Temperatures were measured with thermocouples along the steel bars. The output voltages of the thermocouples were repeatedly scanned in sequence and recorded with a cathode-ray oscilloscope and camera. Power was recorded continuously and the instant of cracking was determined by the parting of a carbon element on the specimen surface. The lengths of cracks were measured immediately after welding.

Temperature distributions were found to be primarily functions of the bar projection (distance of weld from face of concrete). They were not affected much by bar size, thickness of cover, or welding procedure. Higher temperatures obtained with bare bars indicated a considerable amount of conduction of heat to the concrete in the regular units.

Bond tests did not show that any significant decrease in strength was due to the cracks formed by welding. Apparently, with the specimens and procedures used, it made little difference whether the initial crack was formed during welding or later by initial loading in the pull-out test.

Effect of Floor Concrete Strength on Column Strength—Albert C. Bianchini, assistant professor, Clyde E. Kesler, professor, University of Illinois, Urbana, and Robert E. Woods, Indiana State Highway Department, Indianapolis.

Forty-five specimens representing portions of the corner, edge, and interior column and floor sections of a

Three of the five winners of the Wason Medal for Most Meritorious Paper were able to attend the New York convention (left to right): Harry L. Flack and James E. Backstrom, both with the U. S. Bureau of Reclamation, and Richard C. Mielenz, The Master Builders Co. Richard W. Burrows and Vladimir E. Wolkodoff were not present

typical structure were tested under axial compressive loads and the results analyzed to determine the following: (1) how large a differential in column concrete strength and floor concrete strength could be tolerated without decreasing the load-carrying capacity of the column, and (2) the allowable load-carrying capacity of the column if this differential is exceeded. The following variables were included: type of specimen, column concrete strength, and floor concrete strength.

From the analysis of the test results, a procedure was developed for computing the ultimate load of a column in which the column concrete is intersected by floor concrete. These limited tests indicated that the column strength is a function of the ratio of column concrete strength to floor concrete strength and the number of restrained edges tributary to the column. No reduction in column strength occurred for ratios of column concrete strength to floor concrete strength up to 1.4 for all types of specimens and up to 1.5 for most types of specimens.

(This paper appears on p. 1149 of this month's JOURNAL.)

Shear Strength of Restrained Concrete Beams Without Web Reinforcement—John E. Bower, research engineer, Applied Research Laboratory, United States Steel Corp., Monroeville, Pa., and Ivan M. Viest, bridge research engineer, AASHO Road Test, Ottawa, Ill.

Thirty-four beams were tested to determine the effects of a varying moment ratio and a varying moment-shear ratio on the shear strength of restrained reinforced concrete beams.

Fifteen beams failed upon formation of a diagonal tension crack, while 18 beams sustained additional load after formation of the diagonal tension cracks, and eventually failed in shear-compression. One beam failed in flexure without formation of any diagonal tension cracks.

The diagonal tension cracking loads indicated that the moment ratio and the length of the shear span have no direct effect on the diagonal tension cracking load. On the other hand, an increasing moment-shear ratio had the effect of decreasing the load at which the diagonal tension crack forms. An equation, based on the results of the tests and utilizing parameters recommended by ACI-ASCE Committee 326 was presented as being applicable to both simple and restrained beams.

The tests also indicated that the ultimate shear-moment of reinforced concrete beams without web reinforcement is unreliable and, therefore, of academic interest only.

Design and Construction of New York's Pier No. 40—David P. Billington, project manager, Roberts and Schaefer Co., New York.

The new Holland-America Line pier currently under construction on the Hudson River in New York City is de-

signed for both passenger and cargo service. It consists of a main deck of 660,000 sq ft, approximately 800 ft sq in plan, and a two-story peripheral shed approximately 200 ft wide, leaving an open truck court in the center of the pier. The pier has many unusual features but this report was concerned with the concrete main deck and the precast and prestressed concrete superstructure.

Reinforced concrete beams cast monolithically with the concrete pile caps support precast prestressed concrete floor panels which form the main deck.

The main framing for the concrete superstructure consists of: (1) cast-in-place columns, (2) precast columns, (3) precast post-tensioned girders, (4) precast post-tensioned beams, (5) cast-in-place floor slab, and (6) precast roof slab. In addition to the gravity loads, it was required that the superstructure would stand horizontal thrust due to impact when a ship hits the pier.

Annual Research Session

There were 12 progress reports of current research presented at the annual research session on Thursday afternoon. The session is sponsored by ACI Committee 115, Research, as part of its work of reviewing and correlating research in concrete and reinforced concrete. George W. Washa, University of Wisconsin, is committee chairman and S. J. Chamberlin, Iowa State University, is secretary.

This annual open meeting differs from general sessions in that the information is not released for publication. Attendance at the session was the only way of being informed of the developments reported.

Reports of the following research were presented:

Effect of High Temperatures on Concrete Made with Different Aggregates —N. G. Zoldners, head, Construction Materials Section, Minerals Processing Division, Canadian Department of Mines and Technical Surveys, Ottawa, Ont., Canada.

Turbo-Jet Engine Exhaust Impinging Upon Concrete — William R. Lorman, materials engineer, U. S. Naval Civil Engineering Laboratory, Port Hueneme, Calif.

Insulating Concretes — Thomas W. Reichard, Structural Engineering Section, National Bureau of Standards, Washington, D. C.

"High-Low-Lift" Mass Concreting— T. G. Clendenning, supervising engineer, Masonry Section, Research Division, Hydro-Electric Power Commission of Ontario, Toronto, Ont., Canada.

Research Compilation

As part of its work, ACI Committee 115, Research, each year assembles a list of research agencies doing work in the fields of plain and reinforced concrete and various projects in progress. (A summary of the 1960 compilation can be found in the convention report on the annual research session.)

The committee is anxious to increase the coverage of research activities here and abroad. Anyone having suggestions or knowing of omissions from the 1960 compilation should advise the secretary of Committee 115. Information will not be given publicity unless permitted by the correspondent. Address inquiries to:

S. J. Chamberlin
Secretary, ACI Committee 115
T & A M Laboratory
Iowa State University
Ames, Iowa

Anti-Shrinking Agents in Mortar and Concrete—Myron G. Brown, chemical research engineer, Testing and Research Division, Michigan State Highway Department, Lansing.

Effects of Restraint on the Expansion Caused by Alkali-Aggregate Reaction—Thomas B. Kennedy, chief, Concrete Division, Waterways Experiment Station, Corps of Engineers, Jackson, Miss.

Moisture Migration Through Concrete Slab-on-Ground Construction —Harold W. Brewer, development engineer, Products and Applications Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.

Correlation Between Flexural and Tensile Splitting Strength of Concrete —I. Narrow, engineer, Ohio River Division Laboratories, Corps of Engineers, Cincinnati, Ohio.

Admixtures for Concrete

• *By ACI Committee 212*

Concrete admixtures are classified into 11 groups: accelerators, retarders, air-entraining agents, gas-forming agents, cementitious materials, pozzolans, alkali-aggregate expansion inhibitors, damp-proofing and permeability-reducing agents, workability agents, grouting agents, and miscellaneous.

This 34-page report discusses each group and the important effects to be expected in using materials of each group.

Order from Publications Department, American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich.

Price 75¢

Mechanisms of Autogenous Healing in Concrete—Floyd O. Slate, professor of engineering materials, Cornell University, Ithaca, N. Y.

Stress Corrosion Failures in Prestressed Concrete—Mirko Robin Ros, consulting engineer, Zurich, Switzerland.

Investigation of Multiple Panel Reinforced Concrete Floor Slabs—Mete A. Sozen, associate professor of civil engineering, University of Illinois, Urbana.

Development of a Simple Criterion for Determining Ultimate Strength of Columns Subjected to Compression Combined with Unsymmetrical Bending—Boris Bresler, professor of civil engineering, University of California, Berkeley.

Research compilation

Another important part of the work of Committee 115 is the annual compilation of research in progress by various agencies throughout the world. Each convention participant attending the research session was given a copy of this report. A limited supply is available from ACI headquarters for those desiring a copy.

The 1960 compilation lists almost 800 research projects in plain and reinforced concrete. A total of 115 organizations reported research: 42 academic institutions and 31 nonacademic institutions in the United States, and 42 organizations outside the U. S.

Organizations outside the United States reporting research included work going on in many countries: Australia, Canada, Belgium, Denmark, England, France, Germany, India, Italy, Japan, Netherlands, New Zealand, Norway, Portugal, Scotland, South Africa, Switzerland, and Wales.

The compilation is divided into three sections: educational institutions in the United States, noneducational institutions in the United States; and institutions outside the U.S. A subject index is also included.

1960 Officers Elected

The election of Joe W. Kelly as president, Raymond C. Reese as vice-president, and A. Allan Bates, Bruce E. Foster, Ben C. Gerwick, Jr., and James A. McCarthy as members of the Board of Direction was announced.

Mr. Kelly, professor of civil engineering, University of California, Berkeley, succeeds Phil M. Ferguson, University of Texas, Austin, for a 1-year term. Professor Kelly previously served 2 years as vice-president.

ACI's new chief executive has actively participated in Institute affairs since becoming a member in 1926.

He served on the Board of Direction in 1952-54 and 1956-58 and has worked on a number of technical and administrative committees. He is currently a member of the Standards Committee and ACI Committee 611, Inspection of Concrete. He formerly served as author-chairman of Committee 611. During his tenure, the first three editions of the popular *ACI Manual of Concrete Inspection* were produced.

Recognized in engineering circles as a concrete authority, Professor Kelly has authored or coauthored countless technical papers and articles on a wide range of subjects dealing with concrete and concrete materials research.

He was awarded the Wason Medal in 1934 for the paper, "Cement Investigations for Boulder Dam with the Results Up to the Age of One Year," prepared in collaboration with Raymond E. Davis, R. W. Carlson, and G. E. Troxell. In 1946 he received the ACI Construction Practice Award with Bernard D. Keatts for their paper, "Two Special Methods of Restoring and Strengthening Masonry Structures."

Professor Kelly graduated from Purdue University in 1921 and engaged in waterworks engineering for the next 2 years. Then, after a brief term as assistant in the testing materials laboratory at Purdue University, he joined the staff of the Portland Cement Association for 7 years. He returned to Purdue as a concrete specialist of the engineering extension department.

In 1931 he joined the staff of the Engineering Materials Laboratory at the University of California. From associate research engineer, he advanced to the rank of professor of civil engineering in 1939. Later he was named vice-chairman of the department, a position he relinquished in 1958.

Raymond C. Reese gets V-P post

Mr. Reese, who was elected to a 2-year term as vice-president, is a consulting engineer in Toledo, Ohio.

Joining ACI in 1936, he has participated in nearly every phase of Institute activity. Mr. Reese served for 8 years as chairman of ACI Committee 315, Detailing of Reinforced Concrete Structures, which is responsible for the *Manual of Standard Practice for Detailing Reinforced Concrete Structures.* He has been a member of the Standards Committee and is chairman of ACI Committee 318, Standard Building Code.

An authoritative author in the field of reinforced concrete for many years, he received the ACI Lindau Award in 1957 "in recognition of his years of effective coordination of practice and instruction through textbooks and technical papers on reinforced concrete."

Educated in engineering at Massachusetts Institute of Technology and Harvard University, he spent several years as an industrial engineer with the General Electric Co. Since 1922 he has been in private practice as a consulting engineer; he has also been a consultant for the Hausman Steel Co. for over 30 years.

He is author of the *CRSI Design Handbook*, and received the 1953 Award of the Concrete Reinforcing Steel Institute for "his contributions to reinforced concrete construction through design."

Lewis H. Tuthill continues as V-P

In addition to Mr. Reese, Lewis H. Tuthill begins his second year as an ACI vice-president. Mr. Tuthill, elected to a 2-year term in 1959, is a concrete engineer with the Division of Design and Construction, California State Department of Water Resources, Sacramento.

Four directors installed

Elected to 3-year terms on the ACI Board of Direction were A. Allan Bates, Bruce E. Foster, Ben C. Gerwick, Jr., and James A. McCarthy.

Dr. Bates, vice-president for research and development, Portland Cement Association, Chicago, is a scientist and chemical engineer with broad experience in industrial research.

Active in Institute affairs since 1946, Dr. Bates served on the Board of Direction from 1952-54. He was chairman of the ACI Building Committee during 1957-58, the work of which culminated in the new ACI headquarters building.

Dr. Bates holds degrees from Ohio Wesleyan University and Case and a DSc from the University of Nancy in France.

Dr. Foster, assistant chief, Concreting Materials Section, National Bureau of Standards, Washington, D. C., has been an ACI member since 1948. He served on the Board of Direction from 1956-58 and is currently chairman of ACI Committee 212, Admixtures, and a member of the Standards Committee.

Dr. Foster is a graduate of Colorado College, University of Oregon, and Stanford University. He joined the National Bureau of Standards in 1935 and much of his time has been spent in the California acceptance testing laboratories. He was associated with the testing of pozzolan cement for Bonneville Dam and was in charge of the Permanente branch laboratory which tested the cement for Shasta Dam and for most of the Pacific-area construction during World War II.

Mr. Gerwick, president of Ben C. Gerwick, Inc., San Francisco, is a former president of the Prestressed Concrete Institute.

Mr. Gerwick, graduate of the University of California, joined Ben C. Gerwick, Inc., in 1940 for a brief period prior to active military duty. He rejoined the organization in 1946 and since that time has filled positions of field engineer, office engineer, vice-president, and in 1952 became president of the company.

Professor McCarthy, professor of civil engineering, University of Notre Dame, Notre Dame, Ind., joined ACI in 1940 and has been engaged in technical committee work for a number of years. He is currently a member of ACI Committee 115, Research, and a member of the Standards Committee. He was a member of the Technical Activities Committee from 1955-56.

Continuing on the Board of Direction are Arsham Amirikian, Bureau of Yards and Docks, Department of the Navy, Washington, D. C.; S. J. Chamberlin, Iowa State University, Ames; Roger H. Corbetta, Corbetta Construction Co., New York; George C. Ernst, University of Nebraska, Lincoln; E. A. Finney, Michigan State Highway Department, East Lansing; Bryant Mather, Waterways Experiment Station, U. S. Corps of Engineers, Jackson, Miss.; Walter J. McCoy, Lehigh Portland Cement Co., Coplay, Pa.; and Cedric Willson, Texas Industries, Inc., Fort Worth.

The three most immediate past presidents also serving on the Board are Phil M. Ferguson, University of Texas, Austin; Walter H. Price, U. S. Bureau of Reclamation, Denver; and Douglas McHenry, Portland Cement Association, Skokie, Ill.

Nominating Committee

The newly elected Nominating Committee consists of Milo S. Ketchum, Jr., of Ketchum, Konkel & Hastings, Denver; Ernst Gruenwald, manager, Incor and Technical Service, Lone Star Cement Corp., New York; W. C. Hansen, director, Research Laboratories, Universal Atlas Cement Division, U. S. Steel Corp., Gary, Ind.; Eivind Hognestad, manager, Structural Development Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill.; K. B. Woods, head, School of Civil Engineering, Purdue University, W. Lafayette, Ind.

Also serving on the committee will be the three most recent past presidents of the Institute, Messrs. Ferguson, Price, and McHenry.

Honorary Members and Awardees Announced

Honorary membership was conferred on two long-time ACI members and 11 engineers received awards at the annual Awards Luncheon on Wednesday, March 16.

Honorary members

Honorary membership in the American Concrete Institute was conferred on Prof. Charles H. Scholer, Manhattan, Kan., and William Lerch, Chicago. Both were honored for their outstanding contributions to ACI and the concrete industry, especially in the field of research.

They become the 30th and 31st persons to receive the token of esteem that an ACI honorary membership signifies.

Professor Scholer is a consultant and former head of the Department of Applied Mechanics at Kansas State College. He served as president of the American Concrete Institute in 1954 and has long been active in Institute affairs, both in technical and administrative committees.

Eminent in the field of concrete research, Professor Scholer was one of the first to discover the benefits of entrained air in improving the durability of concrete.

Mr. Lerch, who retired in 1959 as assistant to the vice-president for research and development of the Portland Cement Association, is widely known for his authoritative papers on the chemistry of cement, concrete, and air-entrained concrete.

ACI Awards

Winner of the Henry C. Turner Medal was Harrison F. Gonnerman, Oak Park, Ill. The Alfred E. Lindau Award was presented to Arthur R. Lord, Palos Park, Ill. The Henry L. Kennedy Award was awarded to Harry C. Delzell, Chicago.

Two University of California faculty members, Boris Bresler and K. S. Pister, collaborated to win the Wason Medal for Research. In another team effort, five authors pooled their talents to win the Medal for Most Meritorious Paper. They were James E. Backstrom, Richard W. Burrows, and Harry L. Flack, Denver; Richard C. Mielenz, Cleveland; and Vladimir E. Wolkodoff, Niagara Falls, N. Y.

Rounding out the award winners was J. F. Camelleries, New York, who was presented the ACI Construction Practice Award.

As recipient of the Turner Award, Mr. Gonnerman was cited "for numerous and outstanding contributions to concrete technology through research and administration."

Since 1952 Mr. Gonnerman has operated his own consulting engineer-research service. Prior to this, he served for 30 years on the research and development staff of the Portland Cement Association, Chicago.

An active ACI member since 1918, he served as Institute president in 1947. He was awarded honorary membership in 1951. Mr. Gonnerman has twice re-

Retiring ACI President Phil M. Ferguson (left) congratulates Prof. Joe W. Kelly on his election to the ACI presidency for 1960

ceived the ACI Wason Medal for noteworthy research — in 1929 and 1944.

Mr. Lord was awarded the Lindau Award as "originator of reinforced concrete building tests and a pioneer authority in the development of flat slab construction."

Now retired, Mr. Lord's curiosity to know more about the design and control of concrete led to the development of many "firsts" for the industry. He helped develop strain gages for use on concrete structures, and he originated the first actual load test of a building as a basis for stress analysis.

He was active in analysis of flat slab construction and pioneered in a variety of structural designs. During the 1920's he was the world authority on flat slab design.

Mr. Lord has been an ACI member since 1916 and helped found the Institute's official publication, the JOURNAL.

Also a winner of ACI's Wason Medal (1927), he was elected to the ACI presidency in 1934. He was made an honorary member of the Institute in 1957.

The Kennedy Award, in the form of a framed scroll, was conferred upon Mr. Delzell for his 20 years of outstanding service advancing the objectives of the American Concrete Institute.

Mr. Delzell is managing director of the Concrete Reinforcing Steel Institute, Chicago. He was instrumental in ACI's adoption of the comprehensive manual of standard practice on detailing reinforced concrete structures. Mr. Delzell has worked on a number of ACI technical and administrative committees and on the Board of Direction. He participated actively and highly effectively in the fund-raising campaign that financed the construction of the ACI headquarters building in Detroit.

The Wason Medal for Research was shared by Messrs. Bresler and Pister, respectively, professor and associate professor of civil engineering, University of California, Berkeley, for their paper, "Strength of Concrete Under Combined Stresses." The paper was published in the September, 1958, ACI JOURNAL.

The five collaborating authors who received the Wason Medal for Most Meritorious Paper won with their paper, "Origin, Evolution, and Effects of the Air Void System in Concrete." The award-winning paper was published as four installments in the JOURNAL during 1958.

Mr. Backstrom heads the Materials, Mixes and Durability Section of the Concrete Laboratory Branch, U. S. Bureau of Reclamation, Denver. Mr. Burrows is design consultant for the Martin Co., Denver, and was formerly with the engineering laboratories of the Bureau of Reclamation. A third coauthor, Mr. Flack is currently employed by the Bureau of Reclamation, Denver, engaged in research on the freezing and thawing durability of concrete. Mr. Mielenz, director of research, The Master Builders Co., Cleveland, was also formerly with the Bureau of Reclamation. Mr. Wolkodoff is senior engineer and head, physical measurements group, Research and Development Division, The Carborundum Co., Niagara Falls, N. Y. Previously he was with the Bureau of Reclamation.

Mr. Camellerie, vice-president and chief engineer of Plummer Associates, New York, won the Construction Prac-

tice Award for his paper, "Slip Form Details and Techniques." This paper was published in the April, 1959, JOURNAL.

Prof. Joe W. Kelly, newly elected ACI president, also presented a plaque honoring the retiring president to Prof. Phil M. Ferguson.

Local Committee Sponsors Special Events

Augmenting the technical program, the New York Convention Committee arranged an impressive schedule of entertainment, with ladies given prime consideration in the planning.

All conventioners were invited to a get-acquainted social gathering on Tuesday evening.

Two major field trips were arranged. One was a visit to two projects with unusual applications of reinforced and prestressed concrete—the spiral ramped art museum designed by Frank Lloyd Wright, the Guggenheim Museum, and the George Washington Bridge where prestressed concrete work is in progress on the Manhattan approaches. The other trip included Kips Bay housing project — 21-story concrete buildings with cast-in-place framing and walls and exposed columns — and then New York International Airport to view hangars with long span concrete cantilevers.

The ladies' program included coffee hours each morning, a fashion show and luncheon, an uptown tour of Manhattan, United Nations buildings tour and luncheon at the Delegates Dining Room, and a downtown tour of Manhattan.

Local committee

Assisted by a local planning committee of over 70 prominent men, Roger H. Corbetta, president of Corbetta Construction Co., general chairman of the group, rolled out the "red carpet" to make the 56th annual ACI meeting a memorable event. Dugald J. Cameron, Concrete Reinforcing Steel Institute, served as secretary of the committee, with Admiral John J. Manning, USN (Ret.), of the Concrete Industry Board of New York, as treasurer.

Line-up of subcommittee chairmen and members were: Entertainment — M. J. McMillan, consulting engineer, chairman, with all members of the local group assisting on the committee. Publicity — Fred J. Driscoll, George F. Driscoll Co., chairman, aided by Rear Admiral Martin W. Kehart, representing Society of American Military Engineers, and Captain Emil H. Praeger, representing American Society of Civil Engineers.

Membership Promotion—Jacob Feld, consulting engineer, chairman, with W. J. McGinnis, Pratt Institute; Max Abramovitz, Harrison and Abramovitz; Peter R. Foss, W. J. Barney Corp.; Ernst Gruenwald, Lone Star Cement Corp.; and L. D. Long, Gulick-Henderson Testing Laboratories, working on the committee.

Inspection Trips — Elliot A. Haller, Haller Testing Laboratories, chairman, assisted by R. G. Watson, Universal Atlas Cement Co.; James E. Halpin, Transit Mix Concrete Corp.; Gregory E. Brooks, Voorhees, Walker, Smith, Smith and Haines; John M. Kyle, Port of New York Authority; William W. Karl, Lehigh Materials Co.; Frederick S. Merritt, *Engineering News-Record;* Boyd Anderson, Ammann and Whitney; and William H. Mueser, Moran, Proctor, Mueser and Rutledge.

Educational Committee — Brother Amandus Leo, FSC, Manhattan College Engineering School, chairman, working with committee members: Jewel M. Garrelts, Columbia University; Charles E. Schaffner, Brooklyn Polytechnic In-

stitute; James Michalos, New York University; Milton Alpern, Cooper Union; William S. LaLonde, Jr., Newark College of Engineering; Paul Hartman, City College of New York; Joel Wiesenfeld, Rutgers University; Admiral W. Mack Angas, Princeton University; Sidney Borg, Stevens Institute of Technology; James Grote Van Derpool, Columbia University; Esmond Shaw, Cooper Union; Robert W. McLoughlin, Princeton University; Olindo Grossi, Pratt Institute.

Robert K. Lockwood, Sika Chemical Corp., headed up the Exhibits Committee with James Noonan, Praeger-Kavanagh; Dan Morris, Testlab Corp.; and William Griffin, *Engineering News-Record*, as committee members.

George F. Ferris, Raymond International, Inc., chairmanned the Finance Committee assisted by William H. Mueser, Moran, Proctor, Mueser, and Rutledge; H. T. Noyes, Turner Construction Co., Inc.; Admiral John J. Manning, Concrete Industry Board; and Roger H. Corbetta, Corbetta Construction Co., Inc.

Others working on the local planning committee included: W. A. Blaha, Forbes Fireproofing Corp.; Waldo G. Bowman, *Engineering News-Record;* George N. Cohen, representing Concrete Industry Board of New York; Clayton L. Davis, Universal Atlas Cement Co.; William Eipel, Eipel Engineering; A. L. Glassett, W. J. Barney Corp.; Frank Kelly, Colonial Sand and Stone Co.; J. C. McClure, Universal Atlas Cement Co.; William J. McIntosh, representing Concrete Industry Board; A. F. Moore; Penn-Dixie Cement Co.; George F. Pavarini, Pavarini Construction Co., Inc.; Anton Tedesko, Roberts and Schaefer Co., Inc.; Maxwell Upson, Raymond International, Inc.; and Carl Youngdahl, Cement League (BTEA).

Women's entertainment committee was chairmanned by Mrs. Elliot Haller with Mrs. Raymond C. Reese acting as honorary chairman. The committee included: Miss Helen E. Casey, Mesdames Normer L. Gray, William Kinneman, Helen Labahn, Robert K. Lockwood, John J. Manning, M. J. McMillan, H. T. Noyes, George F. Pavarini, Emil Schmid, Miss Rosette Tarricone, Mrs. Anton Tedesko and Miss Thelma Weiss.

Convention Exhibitors

The product displays were one of the focal points of ACI's New York Convention. Twenty-four firms and organizations serving the concrete industry showcased their products and services in 33 booths during the convention. Technical information and advice was yours for the asking . . . from company representatives manning the booths and from a wealth of helpful technical pamphlets and product literature. The line-up of exhibitors included:

Acrow Corp. of America, Carlstadt, N. J.—Display of this world-wide firm specializing in formwork featured actual set-up of adjustable steel shores. Literature depicting uses of Acrow's V-form system of formwork, shores, and scaffolding on different types of construction projects was available.

American Pecco Corp., White Plains, N. Y.—A working scale model of the Pecco construction tower crane highlighted this display. Pecco representatives distributed brochures detailing on-the-job performances of tower cranes. Several different models are available. Guides to the erection and stripping of Pecco horizontal shoring also were distributed.

Autolene Lubricants Co., Denver, Colo. —Backdrop display pointed up the uses

A KALEIDOSCOPE OF HELPFUL PRODUCT DISPLAYS greeted ACI delegates who visited the exhibit hall during the New York convention. Equipment, materials, and services to solve most problems in concrete construction and design were spotlighted by colorful displays. Some of the 24 exhibits are pictured here. ACI's educational exhibit is shown in the center photo

of Protex dispersing agent and Protex air-entraining solution. Literature on other Protex products for curing, sealing, and bonding concrete, Probond epoxy formulated products, and other concrete additives was passed out.

Bethlehem Steel Co., Bethlehem, Pa.—Technical representatives of this supplier of steel bars for concrete reinforcement were on hand to answer questions concerning different bars used in concrete reinforcement. Helpful literature included booklet on the welding of bars.

Chicago Fly Ash Co., Chicago, Ill.—Display was co-sponsored by McNeil Bros., Bridgeport, Conn., Detroit Edison Co., Detroit, Mich., and Walter N. Handy Co., Evanston, Ill. Display pointed up the advantages of fly ash concrete. Available were proportioning guides for concrete mixes containing fly ash.

Concrete Industry Board of New York—Technical data and booklets covering concrete engineering were distributed.

Clipper Manufacturing Co., Kansas City, Mo.—The Clipper line of masonry and concrete cutting saws and blades was featured. Clipper also manufactures diamond core bits and accessories for concrete and masonry drilling.

Elgood Concrete Forms Corp., Brooklyn, N. Y.—Colorful film slide display showed various uses of Elgood inflatable void forms for casting sewers, pipe, and drains. Also depicted were uses of Voidcrete cored slabs and Elgood synchronized jacking systems.

Expanded Shale Clay and Slate Institute, Washington, D. C.—The many aggregates produced by its members were listed in this institutional display. Technical data of value to the concrete engineer and designer were available.

The Fireproof Products Co., Bronx, N. Y.—A display, supplemented by product literature, described this company's line of fireproofing materials and products.

Hugh F. Fenlon, Atlanta, Ga.—Hugh Fenlon, consulting engineer, was on hand to discuss his newly-published book, *Reinforced Concrete Column Tables Ultimate Strength Design.* Book contains 300 tables showing 8568 column designs.

Johns-Manville Products Corp., New York, N. Y.—J-M's Celite Division display spotlighted Placewel, its liquid concrete admixture, and Retardwel, its liquid retarder for concrete. Brochures explained the advantages of both additives in the light of field and laboratory tests.

The Master Builders Co., Division of American-Marietta Co., Cleveland, Ohio—A mobile display featuring construction photos was an attention-getter. Actual uses of Pozzolith and other admixtures and products were pictured, augmented by literature.

The Patent Scaffolding Co., Inc., Long Island City, N. Y.—Miniature scaffolding display called attention to company's line of sectional steel shoring and scaffolding, as used in masonry, highway, bridge, tunnel, and other construction. Descriptive literature was available.

The Prescon Corp., Corpus Christi, Tex.—The Prescon system of prestressing was dramatized in full color slides. Other photos and brochures depicted a wide variety of structures incorporating the Prescon system, which was developed in 1950 and employs the post-tensioning technique.

Raymond Concrete Pile Co., New York, N.Y.—Color slides illustrated the use of cast-in-place concrete piles. Brochures also pinpointed impressive structures around the world that are anchored on Raymond piles. The company's soil investigation services also were explained.

Rex-Spanall, Inc., New York, N. Y.—A nationwide specialist in the concrete forming and shoring field, the Rex-Spanall exhibit emphasized its hori-

zontal shoring techniques and equipment for building and heavy construction. Brochures detailed the Rex-Spanall line of equipment and engineering services.

Richmond Screw Anchor Co., Inc., Brooklyn, N. Y.—Information and literature were available on Richmond's diversified line of construction equipment. Illustrated pamphlets showed the use of lifting inserts, rock anchors, reinforcing bar supports, screed chairs, snay-typs, etc.

Sika Chemical Corp., Passaic, N. J.—An interesting display called attention to such Sika products as Plastiment, Sikacrete, and Rugasol. Technical data were distributed.

Sonneborn Chemical and Refining Corp., New York, N. Y.—Sonneborn's Building Products Division display featured its wide range of products developed for concrete construction. Included were coating and treatments for concrete flooring, waterproofing, dampproofing, concrete and mortar admixtures, caulking and sealing compound, protective coatings, and curing and sealing compounds.

Sonoco Products Co., Hartsville, S. C.—Sonoco fiber tubes and forms were on display. Backdrop of the exhibit featured a pictorial record of concrete construction projects in which Sonovoid and Sonotube were utilized. Also available for inspection was Duropipe, a new bituminized fiber pipe.

Thor Power Tool Co., Aurora, Ill.—Thor power trowels and concrete vibrators were on display. The Thor product line, including vibratory finishing screeds, was also described in literature.

Vibro-Plus Products, Inc., Stanhope, N. J.—The Vibro-Plus electric, pneumatic and gasoline drive concrete vibrators were on display, including a motor-in-head electric vibrator. Literature describing external vibrators and Terrapac soil compactors also was available.

Water Seals, Inc., Chicago, Ill.—The different types of waterstops made by Water Seals were on display. Visitors received a brochure describing these waterstops, their application, and installation.

Technical Committee Appointments

At the 1960 annual meeting in New York, the Board of Direction approved the appointment of 12 new committee chairmen for 3-year terms, with the former committee chairmen remaining as members of the respective committees.

Committee 115, Research

Chester P. Siess, professor of civil engineering, University of Illinois, Urbana, succeeds George W. Washa, professor and chairman of the department of mechanics, University of Wisconsin, Madison.

Committee 116, Nomenclature

Delmar L. Bloem, associate director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D.C., succeeds Richard C. Mielenz, director of research, Master Builders Co., Cleveland.

Committee 215, Fatigue of Concrete

John F. McLaughlin, research engineer, Purdue University Engineering Experiment Station, Lafayette, Ind., succeeds Gene M. Nordby, head, department of civil engineering, University of Arizona, Tucson.

Committee 326, Shear and Diagonal Tension (ACI-ASCE)

Eivind Hognestad, manager, Structural Development Section, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., suc-

ceeds the late Charles S. Whitney of Ammann and Whitney, New York.

Committee 331, Structures of Concrete Masonry Units

A. T. Hersey, director of technical service, Alpha Portland Cement Co., Easton, Pa., succeeds S. H. Westby, technical advisor, Housing and Cement Products Bureau, Portland Cement Association, Chicago.

Committee 338, Torsion

Gordon P. Fisher, professor of civil engineering, Cornell University, Ithaca, N.Y., succeeds George C. Ernst, professor of civil engineering, University of Nebraska, Lincoln.

Committee 609, Consolidation of Concrete

Elmo C. Higginson, U. S. Bureau of Reclamation, Denver, succeeds Joseph J. Waddell, chief materials engineer, Knoerle, Graef, Bender and Associates, Chicago.

Committee 612, Recommended Practice for Curing Concrete

Edward E. Bauer, professor of civil engineering, University of Illinois, Urbana, succeeds G. E. Burnett, engineer, U. S. Bureau of Reclamation, Denver.

Committee 616, Coatings for Concrete

William H. Kuenning, senior development engineer, Research and Development Laboratories, Portland Cement Association, Skokie, Ill., succeeds John L. Rohwedder, engineer, Rock Island District, U.S. Army Corps of Engineers, Rock Island, Ill.

Committee 714, Concrete Bins and Silos

William L. Clark, Jr., consulting engineer, Waterville, Ohio, succeeds C. H. Scholer, consultant and professor of applied mechanics, Kansas State College, Manhattan, Kan.

Committee 805, Application of Mortar by Pneumatic Pressure

Thomas J. Reading, engineer, U. S. Army Corps of Engineers, Missouri River Division, Omaha, Neb., succeeds W. G. Irmscher, district sales manager, Peerless Cement Co., Detroit.

One-year reappointments

The following reappointments as technical committee chairmen were approved for 1-year terms because of committee work in progress:

Committee 212, Admixtures

Bruce E. Foster, materials engineer, Concreting Materials Section, National Bureau of Standards, Washington, D.C.

Committee 318, Standard Building Code

Raymond C. Reese, consulting engineer, Toledo, Ohio

Committee 328, Limit Design (ACI-ASCE)

Alfred L. Parme, head, Advanced Engineering Group, Portland Cement Association, Chicago

Committee 333, Design and Construction of Composite Structures (ACI-ASCE)

Ivan M. Viest, bridge research engineer, AASHO Road Test, Ottawa, Ill.

Committee 334, Concrete Shell Structures

Anton Tedesko, vice-president, Roberts and Schaefer Co., New York

Committee 401, Specifications for Structural Concrete

George H. Nelson, president, Law Engineering Testing Co., Atlanta

Committee 622, Formwork for Concrete

Harry Ellsberg, structural consultant, Giffels and Rossetti, Detroit

Committee 712, Precast Structural Concrete Design and Construction (ACI-ASCE)

Jack R. Janney, The Engineers Collaborative, Des Plaines, Ill.

Committee 716, High Pressure Steam Curing

Samuel B. Helms, research engineer, Lehigh Portland Cement Co., Coplay, Pa.

NSA schedules 1960 annual convention

The executive committee of the National Slag Association has scheduled the 1960 annual meeting for November 3-4 at the Hotel Mayflower, Washington, D.C.

Why SONOTARD was selected for New Pier 40; New York's Largest Pier now under construction

Holland-American Line, New York City
Dept. of Marine and Aviation
Vincent A. G. O'Connor, Commissioner
Capt. Lewis H. Rabbage, Chief Engineer

Roberts & Schaefer Co.—Engineers
Stock Construction Corp.—General Contractor
Corbetta-Precrete—Prestress Contractors
Colonial Sand & Stone Co.—Ready Mixed Concrete

• **SONOTARD** is being used in *all* concrete to produce consistent and uniformly high strengths required for the heavy live loads to be handled, plus voidless density to resist erosion from sea water.

• A high degree of workability to facilitate placement has been maintained through high summer heat and a winter which recorded a low mark for sustained cold weather.

• The quantities of **SONOTARD** used were varied according to temperature to keep the water content and rate of hardening constant. At all temperatures **SONOTARD** produced a uniformly high strength with maximum density and eliminated shrinkage and cracking.

• **SONOTARD** played an essential role in the prestressed deck slabs and beams by controlling the initial rate of hardening and fluidity of concrete for optimum placement properties.

• **SONOTARD** speeded the strength development (15 hours of steam cure averaged over 4500 psi) permitting early release of tension and early stripping of forms.

• Since **SONOTARD** does not increase the air content while reducing water requirements it allows the operator to control accurately the total air at all times.

Consult your local SONNEBORN representative for your particular project to learn how **SONOTARD'S** exclusive formulation can help you produce harder, denser floors—more uniform, crack resistant structures—water-tight concrete of minimum shrinkage.

SONNEBORN
CHEMICAL AND REFINING
CORPORATION
Building Products Division, Dept. J50
404 Park Avenue South, New York 16, N. Y.

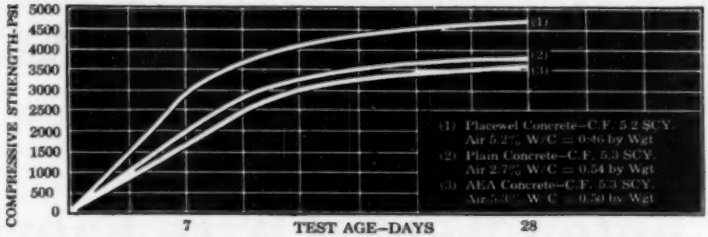
Increase concrete strength

30% with J-M Placewel...

the liquid admixture that gives you 5 key controls of concrete

Placewel®, a product of Johns-Manville, world's largest industrial user of portland cement, permits a new high in compressive and flexural strength. This superiority has been proved in millions of yards of concrete.

Placewel is a water-reducing, dispersing agent *plus* a catalyst. It is available with or without an air entraining agent. Placewel, by breaking up the cement flocs, releases water for lubrication that isn't normally available. Thus, mixing water requirements are cut without relying on air entrainment alone. The Placewel catalyst enhances the strength-gaining characteristics of portland cement. This unique combination of properties provides a 30% increase in concrete strength (see chart).



Compressive strength — performance of Placewel vs. plain and air entrained concrete

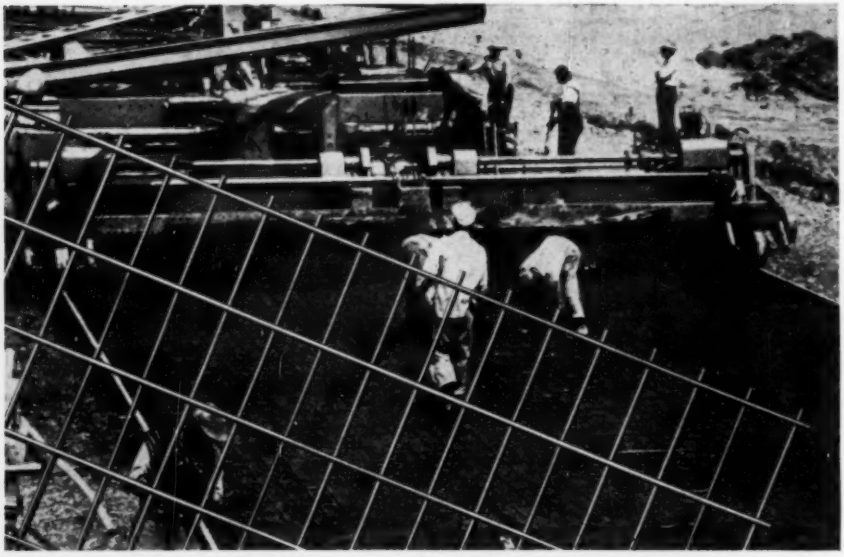
Less than three ounces of Placewel per sack of cement will also —

- Increase durability 350%
- Increase workability
- Improve dimensional stability
- Reduce permeability

J-M Retardwel® is recommended when concreting conditions require longer placing and finishing time. Retardwel delays the initial set of concrete and yet provides higher 24-hour strength.

Architects and engineers the world over have learned that Placewel and Retardwel give them complete control over the concrete they specify. For full information and technical assistance write: Johns-Manville, Box 14, N. Y. 16, N. Y.

JM PRODUCT JOHNS-MANVILLE *Celite Division*



Sinews of steel
to strengthen concrete construction

Clinton Welded Wire Fabric

For years, CF&I has worked closely with the construction industry, developing strong welded steel mesh for reinforcing purposes. The result: CF&I-Clinton Welded Wire Fabric.

This rugged mesh gives concrete structures—from bridges and roads to culverts and skyscrapers — maximum strength and durability. Its dependability stems from the high quality of the steel wire used.

CF&I-Clinton Welded Wire Fabric is manufactured to all ASTM specifications and is furnished in either rolls or mats. It is supplied in a wide range of gages and spacings.

For complete information, call our nearest sales office.

CLINTON
WELDED WIRE FABRIC
THE COLORADO FUEL AND IRON CORPORATION

CF&I STEEL

In the West: **THE COLORADO FUEL AND IRON CORPORATION**—Albuquerque • Amarillo • Billings • Boise Butte • Denver • El Paso • Farmington (N. M.) • Ft. Worth • Houston • Kansas City • Lincoln • Los Angeles Oakland • Oklahoma City • Phoenix • Portland • Pueblo • Salt Lake City • San Francisco • San Leandro Seattle • Spokane • Wichita

In the East: **WICKWIRE SPENCER STEEL DIVISION**—Atlanta • Boston • Buffalo • Chicago • Detroit New Orleans • New York • Philadelphia

CF&I OFFICE IN CANADA: Montreal
CANADIAN REPRESENTATIVES AT: Calgary • Edmonton • Vancouver • Winnipeg

7520

Positions and Projects

Change in title and mission of ACI Committee 209

The title of ACI Committee 209, formerly "Volume Changes and Plastic Flow in Concrete," has been changed to "Creep and Volume Changes in Concrete."

The mission of the committee has been revised to read: "The assignment of Committee 209 is to review critically available data on the factors affecting the magnitude of creep and volume changes of concrete; to suggest or recommend topics of needed research; to stimulate the preparation of papers; and to interpret available information in a form that is useful to the designer."

ACI headquarter's visitors

Well over a hundred visitors have inspected the ACI headquarters building in Detroit recently.

A group of 54 architects from the Finnish Architects Institute, Finland, visited the building early in April. Student tours from the University of Minnesota and University of Illinois also included the ACI headquarters in their itinerary.

In addition to a number of Michigan and Ohio guests, the visitors' register shows engineers and architects from Guatemala, Canada, and Japan.

ACI Michigan members meet

About 40 ACI members and guests from the Michigan area met for a luncheon meeting on March 9, with Clem G. Merriman, chief of the civil division, General Engineering Department, Detroit Edison Co., Detroit, as the guest speaker.

Mr. Merriman, member of the recently reactivated ACI Committee 505, Design and Construction of Reinforced Concrete Chimneys, discussed briefly problems which have arisen in recent years in the use of tall chimneys or stacks by utility companies. Problems highlighted included structural failures due to aerodynamic effects and also the deterioration of the linings and shells from the development of corrosive acids within the chimney.

The formal petition for the activation of a Michigan Chapter of ACI was approved by the Board of Direction of the Institute at the annual convention in New York City. In addition to the entire state of Michigan, it has been decided to include in this chapter Lucas County of Ohio in which Toledo is the most prominent city. People from the latter area have been most active with the Michigan members and it is hoped that these geographical boundaries will encompass an active ACI group.

Szilard addresses NSPE in Seattle

Rudolph Szilard, structural design specialist, Martin Co., Denver, was principal speaker for "Engineers Week" activities on February 22-26 in Seattle. Sponsored annually by the National Society of Professional Engineers, the week-long conference was hosted by the Puget Sound Engineering Council.

Over 400 representatives of 28 technical societies comprising the Puget Sound Council heard Mr. Szilard speak on the " General Problems of a Lunar Base Design." His talk was based on research work done by Mr. Szilard for the Martin Co.'s Space Medicine Section in cooperation with M. G. Galmish, principal engineer of the company's Ground Base System's Development Department and Dr. J. Gaume, chief of Martin's Space Medicine Section.

In his present post Mr. Szilard is responsible for the utilization of underground double-curved thin shells for

Titan bases. Prior to joining the Martin organization as a design specialist, he was associated with Ammann and Whitney, New York, and most recently was an associate professor at Colorado State University, Fort Collins. Mr. Szilard is a member of ACI Committee 338, Torsion.

Prestressed Concrete Institute schedules 6th annual meeting

The 6th annual convention of the Prestressed Concrete Institute will be held September 27-30 at the Statler-Hilton Hotel in New York City.

Under the theme: "Prestressed Concrete — Key to Creative Architecture and Imaginative Engineering," the technical program will feature top engineers from the United States, Canada, Europe, and Russia who will present papers on the latest methods and projects utilizing prestressed concrete.

Each day of the convention will provide one general session for all attendees. During this session new concepts and techniques of design and execution will be discussed. Separate sessions will be provided for the analysis and study of methods and problems pertinent to architects, engineers, and producers.

Leap Associates announce opening of Atlanta plant

The opening of Leap Structural Concrete of Atlanta represents the 55th prestressed concrete operation under Leap consultantship. Located at Powder Springs, Ga., with offices in Atlanta, the plant will supply that area with roof and floor building sections. The plant will be managed by J. Warren Thompson with John E. LaRowe as sales manager.

Galezewski elected Rockwin vice-president

Steven Galezewski, chief engineer of Rockwin Prestressed Concrete Corp., Santa Fe Springs, Calif., has been elected a vice-president of the firm. Mr. Galezewski has been associated with Rockwin since its establishment and is a "pioneer" in prestressed concrete in this country.

Active in ACI and a number of other technical societies, Mr. Galezewski is currently serving as a foreign magazine reviewer for the Current Reviews section of the JOURNAL.

Riverside Cement announces new multi-million dollar facility

John M. Kinard, president of Riverside Cement Co., division of American Cement Corp., recently announced the awarding of a contract to Diversified Builders, Inc., Los Angeles, for construction of a $5 million white portland cement manufacturing facility at the company's Crestmore plant near Riverside, Calif. The plant will be the only facility producing white cement west of the Rocky Mountains. The plant will have an initial annual capacity of 250,000 bbl, expandable to more than double that amount.

Errata

The following corrections should be made in "Design of Unsymmetrical Reinforced Concrete Sections," by A. Siev, which was published in the April, 1960, ACI JOURNAL.

p. 1064—in Eq. (11), the denominator should be changed from A_{j-d} to $A\,j\,d$.

p. 1067 — in the equation directly above Table 1, change

$$d = \frac{d\,I_{xy}}{j\,d\,s_x - I_{xy}} = \ldots$$

to

$$\propto d = \frac{d\,I_{xy}}{j\,d\,s_x - I_{xy}}$$

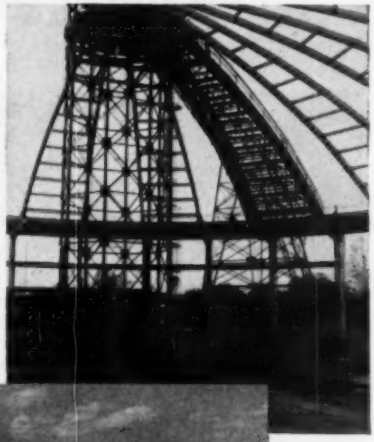
p. 1069—in the second equation from the top of the page, change the number to read $jd = 8.94 + m = 10.815$ in. On the same page and two equations below the above mentioned equation, change the numbers to read $M = jdc = 10.815 \times 6320 = 68{,}500$ in.-lb.

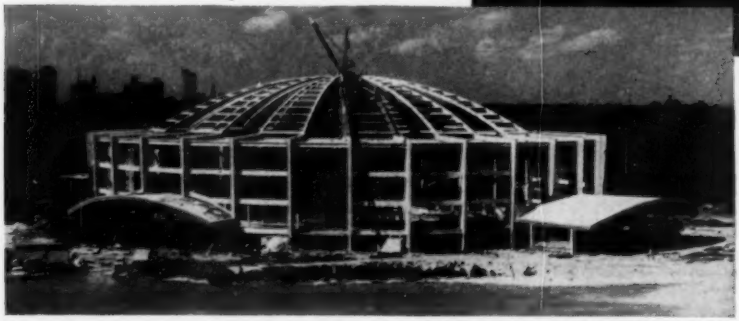
CONCRETE FRAME FOR NEW JACKSONVILLE COLISEUM

Lehigh Early Strength Cement Used in Unique Roof Structure

• Rising next to the famed Gator Bowl in Jacksonville, Florida, is a reinforced concrete frame providing a support free area 302′ in diameter for the new 12,000 seat Sports Coliseum. Lehigh Early Strength Cement and Lehigh Portland Cement are being used throughout.

A concrete ring 25′ in diameter was poured first atop the center support of this huge two-pronged form. Then the circular roof ribs were poured in pairs in prong forms (only one visible), tying the center ring and outer roof together. After each pour, prongs were lowered 3′ hydraulically and rotated to next position.





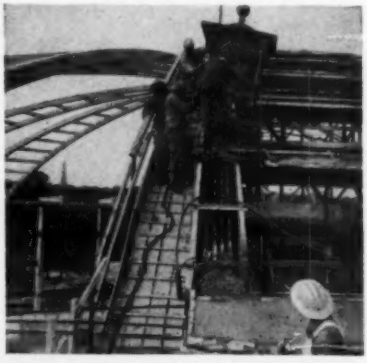
Clear span at base of structure is 302′. Height to center roof ring is 95′. Height to perimeter roof is 65′. Circular roof rib radius is 250′. Two of three entrances take shape in foreground.

Use of Lehigh Early Strength Cement in the perimeter roof made it possible for Daniel Construction Company to use fewer sets of forms while meeting a fast construction schedule. In the circular roof ribs, this cement provided the high early strength concrete necessary for most efficient use of the huge two-pronged form (see pictures).

This is another example of why we say, "Somewhere on nearly every job, Lehigh Early Strength Cement can save time and money." Lehigh Portland Cement Company, Allentown, Pa.

LEHIGH CEMENTS

Owner: City of Jacksonville, Florida
Architect: A. Eugene Cellar and George Ryad Fisher, Jacksonville, Fla.
Structural Engineer: Gomer E. Kraus, Jacksonville, Fla.
Contractor: Daniel Construction Company of Florida, Jacksonville, Fla.
Ready Mix Concrete: Southern Materials Company of Florida, Jacksonville, Fla.



Placing concrete for roof ribs high above coliseum floor.

New, improved USS American Welded strength of 75,000 psi... and a

Wire Fabric has minimum tensile minimum yield point of 60,000 psi...

to give you a better, stronger product at no increase in cost!

For more than 50 years, USS American Welded Wire Fabric has been doing an outstanding job of reinforcing all kinds of concrete work – from porches and walks to skyscrapers and highways. And now–because of its greatly increased tensile and yield strength–it will give even greater strength, longer life, increased freedom from cracking and less maintenance. Also, it will permit longer joint spacing for reinforced slabs on ground or less steel if present joint spacing is used. The new improved Welded Wire fabric will have a 75,000 psi minimum ultimate tensile strength with a minimum yield point of 60,000 psi.

Closely controlled laboratory tests show that if the conventional bond stress theory is applied to American Welded Wire Fabric's resistance to slip, fantastically high bond stress values of from 1,000 psi to 2,700 psi are computed. (See ACI Proceedings, Vol. 48, April, 1952.) Continuing bond test research under the direction of American Iron & Steel Institute has shown such good mechanical anchorage in the concrete as to permit this increase in Tensile Strength of Fabric. American Steel & Wire is able to present this new product because of the tested bond values which enable designers to take advantage of a higher fabric yield point.

Just one example of the advantages of this improved fabric is in one-way slabs. The ACI Building Code 318-56 will allow unit tensile strength for fabric in main reinforcement of 30,000 psi in one-way slabs of 12-foot span or less, provided reinforcing members are ⅜" or less. Previously, designers were limited to 28,000 psi working stress with fabric, and only 20,000 psi with intermediate grade bars.

The new Welded Wire Fabric will cost no more. It will come in the same prefabricated rolls or sheets for easy handling and placing. Therefore, to get the improved product on your job at no extra cost, be sure to specify USS American Welded Wire Fabric.

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James A. Murray

James A. Murray, associate professor in the department of civil and sanitary engineering, Massachusetts Institute of Technology, Cambridge, died recently in Winchester, Mass.

Professor Murray was graduated from Harvard University in 1924. In 1926 he joined the staff of the National Bureau of Standards and was chief of the Lime and Gypsum Section before becoming associated with the Warner Co. of Philadelphia in 1930 as director of research. He joined the MIT faculty in 1948 to take charge of research on cementitious materials and masonry units.

Professor Murray was a member of ACI, American Chemical Society, American Institute of Chemical Engineering, and chairman of the committee on lime for the American Society for Testing Materials.

J. J. Polivka

J. J. Polivka, consulting structural engineer of Berkeley, Calif., died recently at the age of 73.

Dr. Polivka was born in Czechslovakia in 1886. He received a master's degree at Technical University of Prague in 1909, and the degree of Doctor of Technical Science in 1917; he also pursued graduate study at the Federal Institute of Technology, Zurich, Switzerland.

He designed and supervised the construction of many notable structures in Europe, including buildings, dams, and bridges; and became recognized as an authority on mathematical and experimental stress analysis, particularly photoelasticity. Some of his work was in association with such outstanding authorities as Emperger and Mesnager. He designed the Czech pavilion at the Paris International Exhibit in 1937.

In 1939, Dr. Polivka came to the United States to supervise construction of the Czech pavilion, which he had designed, for the New York World Fair of 1939; he then settled in California and became a U. S. citizen in 1945. He was a research associate at the University of California 1939-45, and a lecturer at Stanford University since 1951. He continued his consulting practice by designing unusual structures and consulting on special features of such structures as the Guggenheim Museum in New York, the Johnson Tower in Racine, Wis., and the "butterfly bridge" which has been proposed for crossing the southern arm of San Francisco Bay. In much of his practice he was associated with the noted architect Frank Lloyd Wright.

Winner of a number of competitions and awards, a member of many professional technical societies, Dr. Polivka was author of numerous books, technical papers, and discussions.

A member of the Institute since 1938, Dr. Polivka had been a long-time reviewer for the Current Reviews section of the JOURNAL. A fluent linguist he reviewed technical magazines from Bulgaria, Czechoslovakia, Denmark, France, Germany, Italy, The Netherlands, Poland, Roumania, Russia, Spain, and Yugoslavia.

Leonard C. Urquhart

Leonard C. Urquhart, structural engineer, and since 1949 a member of the firm of Porter, Urquhart, McCreary and O'Brien, consulting engineers, Newark, N.J., died recently in Short Hills, N.J.

Mr. Urquhart received his engineering degree from Cornell University in 1909. He was on Cornell's faculty from 1911 to 1946, serving as professor of structural engineering from 1925.

In 1925 he also was professor of civil engineering at Drexel Institute, Philadelphia, and in 1940 professor of engineering at the University of Hawaii.

In World War I, Mr. Urquhart served as a second lieutenant of engineers. In World War II, on leave from Cornell, he was a colonel in the Corps of Engineers. He was chief of the Engineering

Division, Office of Chief Engineers, Washington, and also served as chief engineer on the Tripler Army Hospital in Hawaii. He received the Legion of Merit.

His firm's projects included Army air bases in Morocco and parts of the New Jersey turnpike.

Mr. Urquhart was the author of *Design of Concrete Structures*, 1940; *Stresses in Simple Structures*, 1932; and *Design of Steel Structures*, 1930. He was editor in chief of the *Civil Engineering Handbook*, 1940. Another of his works was *Elementary Structural Engineering*, 1941.

Mr. Urquhart was a long-time member of ACI and played an active role in ACI Committee 318, Standard Building Code.

Concrete Joint Institute announces new officers

Concrete Joint Institute, formerly known as Expansion Joint Institute, has announced the election of Harry G. Meadows of W. R. Meadows, Inc., Elgin, Ill., as president of the organization.

Other officers elected at the recent meeting of their board of directors were James C. Whitney of Celotex Corp., Chicago, vice-president; and Wallace F. Fischer, Servicised Products Corp., Chicago, treasurer.

Industrial building exposition scheduled for New York

The most extensive conference ever staged in the field of industrial building, the First Industrial Building Exposition and Congress, will be held in the New York Coliseum, December 12-15, under the sponsorship of a board composed of 28 of the nation's foremost architects, builders, engineers, and industrial executives.

Nineteen basic topics dealing with virtually every major aspect of new construction and modernization are scheduled. Discussions will include such subjects as: types of plants to meet specific needs; modernization and expansion; trends in plant design; prefabricated components; advances in the use of concrete; planning a modernization program; and construction problems abroad.

Feibusch forms consulting firm

Hans A. Feibusch has formed his own consulting engineering firm with offices in San Francisco. The firm will specialize in consulting for contractors on heavy construction problems and special equipment, as well as equipment for the manufacturing of prestressed concrete products.

For the past 8 years, Mr. Feibusch has been with the prestressed concrete and foundation firm of Ben C. Gerwick, Inc., San Francisco.

Thürlimann accepts faculty post in Zurich

Bruno Thürlimann, research professor of civil engineering at Lehigh University for the past 7 years, has been named professor of structural engineering at the Swiss Federal Institute of Technology in Zurich.

A native of Switzerland, Dr. Thürlimann made the first design of a complete testing installation in the United States for static and fatigue loading of specimens, components, and entire structures. This installation has seen extensive service in the Fritz Engineering Laboratory at Lehigh University and has been the pattern for other installations that have since been built in other parts of the country.

Dr. Thürlimann joined the Lehigh faculty in 1952 as assistant professor of civil engineering. He was promoted to associate professor of civil engineering in 1955, and to research professor of civil engineering in 1958.

Dr. Thürlimann is a member of ACI-ASCE Committee 333, Design and Construction of Composite Structures, and ACI Committee 334, Concrete Shell Structures.

Irrigation-drainage congress scheduled for Madrid

The 4th International Congress on Irrigation and Drainage will convene May 30 to June 5 in Madrid, Spain, with study tours scheduled for June 6-13.

Among the subjects to be discussed at the working sessions are: reclamation of waterlogged and marshy lands; sprinkler irrigation and comparison with other methods of irrigation; tolerance of plants to minerals in solution in irrigation water and in soil; and use of longitudinal embankments or levees as flood protection measures. Simultaneous interpretation in the three official languages of the congress (English, French, and Spanish) will be provided.

In addition to the excursions included in the program of events, at the close of the congress five different trips are available to tour some of the major irrigation works in Spain.

MIT honors Glasset

Alfred T. Glasset, prominent engineer and builder, recently was awarded the Silver Stein Award for outstanding leadership to his alma mater, the Massachusetts Institute of Technology. He is currently president of the Industrial construction firm of W. J. Barney Corp., Groton, Conn.

Skilken establishes two branch firms

The Skilken Co., Columbus, Ohio, is a new corporation organized to sell the combined services of the Morris Skilken Construction Co., Inc., and the recently formed Skilken Engineering Associates.

Morris Skilken will head the new firm and will also serve as president of the construction firm. The new division of the company, Skilken Engineering Associates, will be headed by B. Lee Skilken, vice-president.

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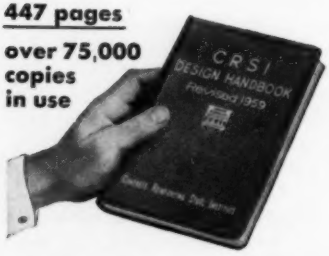

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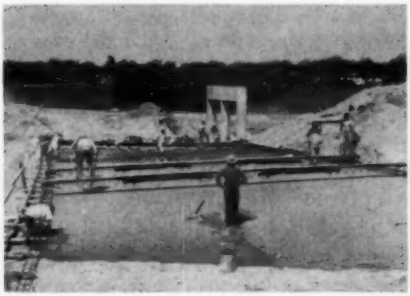

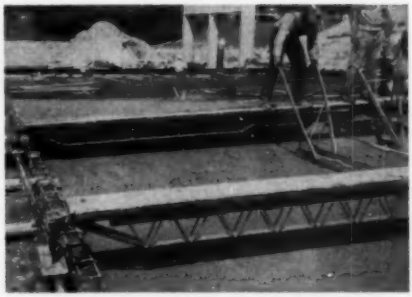

Mil-Wis Construction Co. conceived a unique use of Acrowspan adjustable horizontal shoring (ordinarily used to support formwork and scaffolding) in placing a bridge deck on the Milwaukee Expressway. The units were made into a movable bridge screeding device 45 ft long. Planks, 2 x 14 in., were nailed to the top of the beam from which the finishers operated the longitudinal bull floats. The units rested on pipe supported on the deck panels

Spring Creek power conduit construction

Invitations were issued recently to contractors to bid on construction on the Spring Creek power conduit on the Central Valley project in northern California, according to Grant Bloodgood, assistant commissioner and chief engineer of the Bureau of Reclamation.

The conduit is to be constructed on the project's Trinity River Division near Redding. The contractor awarded the contract for the conduit will be allowed 975 days to complete construction. Bids are to be opened May 5 by the Bureau's project construction engineer at the Trinity River project office, Lewiston, Calif.

The conduit, including a 17-ft diameter, 3100 ft long siphon, will have a total length of about 3 miles. It will comprise two 18.5-ft diameter tunnels, one 8200 ft long and the other 4400 ft long. Major quantities for construction include 181,000 cu yd of excavation for the tunnels, 350,000 cu yd of excavation in open cut, 149,000 cu yd of excavation for the siphon, 60,000 cu yd of concrete and 4,000,000 lb of reinforcing steel for the tunnel linings, and 4,500,000 lb of permanent steel supports.

The tunnels and siphon will operate under pressure and have a capacity of 3600 cu ft per sec. The siphon will be constructed either as a steel-lined concrete structure or as a plate-steel conduit. For the first alternative, there is required 2,000,000 lb of steel-plate liner, 4,500,000 of reinforcing steel, and 10,000 cu yd of concrete. The second alternative for the siphon will require 7,000,000 lb of plate steel.

Spring Creek power conduit will be the final link in the 14 mile long tunnel system conveying water from the Trinity River watershed to the Sacramento River watershed. The conduit will convey water from the division's Whiskeytown reservoir to the Spring Creek powerplant; both features are planned for early construction. Water discharging from the powerplant will enter the existing Keswick reservoir on the Sacramento River.

Hawley named vice-president

Alton J. Hawley has been named vice-president in charge of industrial construction contracts for Fletcher-Thompson, Inc., New York.

Mr. Hawley joined the firm in 1928, immediately after graduation from Pratt Institute. For the past 5 years,

Mr. Hawley has concentrated on modern industrial designs and one of the firm's latest projects is the Syracuse Industrial Park, consisting of 160 acres located at Syracuse, N.Y., where Fletcher-Thompson's staff planned buildings for such companies at this development as General Electric, Pepsi-Cola, Graybar, Continental Can, Nichols Business Machine, and Whitehead Metal Products, plus many others.

A proponent of the tilt-up method of concrete construction, Mr. Hawley has helped pioneer this process throughout the East. He has lectured and written extensively on the subjects of precast concrete and tilt-up construction.

Alpha announces three promotions

Alpha Portland Cement Co., Easton, Pa., has promoted three men to newly created and financial posts.

James I. Maguire has been appointed director of management services; Victor W. Anckaitis, director of property management; and Robert E. Hartmann, controller.

Withey joins Crane staff

Norman H. Withey recently joined Carl C. Crane, Inc., consulting engineering firm in Madison, Wis., to work largely on a special branch of consulting service — concrete technology and engineering. His work will include concrete mix proportioning and control testing for concrete products manufacturers, and review of the concrete and masonry portions of specifications and plans for architects, engineers, or contractors.

Electrical precipitation seminar scheduled at Pennsylvania State

The Pennsylvania State University's annual Electrical Precipitation Seminar is scheduled for June 14-19.

The program is designed for engineers in the public utilities and the cement industry, as well as for other industries concerned with the installation of electric precipitation equipment for smoke and dust abatement.

The seminar will include a study of basic principles, including the nature of dust particles, the electrical charging of particles, the collection and removal of charged particles, and air pollution control.

Vranich forms consulting firm

Formation of the consulting engineering firm of Emil F. Vranich and Associates, Inc., has been announced by Emil F. Vranich, formerly president of Collings-Vranich and Associates, Inc., which has terminated practice. The firm will headquarter in Milwaukee rendering consulting services to architects and industry throughout the Milwaukee and Wisconsin areas.

Bibliography on Prestressed Concrete

This 102-page book (1955 edition) lists 2100 references to American and foreign papers concerning prestressed concrete, published from 1896 through 1954. Also listed are some 60 United States, British, German, and French patents. The 8½x11-in. format is punched for easy filing in three-ring binders. Supplement to 1954 edition is available for 35 cents.

PRICE $2.00

Order from Publications Department, American Concrete Institute, P.O. Box 4754, Redford Station, Detroit 19, Mich.

Zonolite names Grimm to Chicago post

Clayford T. Grimm was recently appointed manager of the architectural and engineering products department of Zonolite Co., Chicago. Mr. Grimm joins Zonolite after 10 years as assistant director of engineering and technology at the Structural Clay Products Institute, Washington, D.C.

Krasnick named Switzer distributor

Max A. Krasnick of San Diego has been named exclusive distributor in the San Diego County of Switzer panel products for building construction. The company manufactures a modular wall unit of lightweight, precast concrete suitable for bearing or nonbearing walls in all types of structures.

Wing opens consulting engineering office

Wayman C. Wing has opened his consulting engineering office in New York City. Formerly an associate of Seelye, Stevenson, Value and Knecht, New York consulting engineers, he is a registered engineer in New York, California, and Pennsylvania. During the past 12 years with the Seelye firm, Mr. Wing was in charge of the structural design of a number of major projects.

Marsh named president of Permanente Cement

Wallace A. Marsh is the newly appointed president and chief executive officer of Permanente Cement Co., Oakland, Calif., and its subsidiaries, including Kaiser Gypsum Co., where he began his career in 1937 as a sand and gravel salesman.

Mr. Marsh, a former California state highway engineer, became general manager of Permanente in 1949 and 2 years later was elected vice-president of the company.

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LOOKING AHEAD

May 4-8, 1960 — Cleveland Convention, American Society of Civil Engineers, Hotel Cleveland, Cleveland, Ohio.

May 5-6, 1960 — Structural Mechanics Conference, American Society of Civil Engineers, Purdue University, Lafayette, Ind.

May 20-29, 1960—Pan American Highway Congress, Bogota, Colombia

May 30-June 4, 1960—Annual Meeting, Concrete Reinforcing Steel Institute, The Greenbrier, White Sulphur Springs, W. Va.

May 30-June 5, 1960—Fourth Congress, International Commission on Irrigation and Drainage, National Syndicate Headquarters, Madrid, Spain

June 16-22, 1960—Third International Congress of Precast Concrete Industry, Sankt Erikshallen, Stockholm, Sweden

June 20-22, 1960—FIP-RILEM Symposium on Injection Grout for Prestressed Concrete, Norges Tekniske Hogskole, Trondheim, Norway

June 20-24, 1960 — Annual Meeting, American Society of Engineering Education, Purdue University, Lafayette, Ind.

June 26-July 1, 1960 — Annual Meeting and Apparatus Exhibit, American Society for Testing Materials, Chalfonte-Haddon Hall, Atlantic City, N. J.

June 27-July 1, 1960—Sixth Congress, International Association for Bridge and Structural Engineering, Stockholm, Sweden

July 10-22 and July 24-August 5 — Engineering Seminars on Atomic Shelters, Pennsylvania State University, University Park, Pa.

July 11-18, 1960 — Second World Conference on Earthquake Engineering, Science Council of Japan, Tokyo and Kyoto, Japan.

July 17-21, 1960—RILEM International Symposium on Concrete and Reinforced Concrete in Hot Countries, Israel Institute of Technology, Haifa, Israel

Aug. 15-26, 1960 — Summer Institute on Nondestructive Testing, Sacramento State College, Sacramento, Calif.

Sept. 27-30, 1960 — 6th Annual Convention, Prestressed Concrete Institute, Statler-Hilton Hotel, New York, N.Y.

Oct. 10-13, 1960 — American Mining Congress, Metal Mining and Industrial Minerals Convention and Exposition, Las Vegas, Nev.

Oct. 31-Nov. 1-2, 1960—13th Regional Meeting, American Concrete Institute, Pioneer Hotel, Tucson, Ariz.

Nov. 3-4, 1960—Annual Meeting, National Slag Association, Hotel Mayflower, Washington, D.C.

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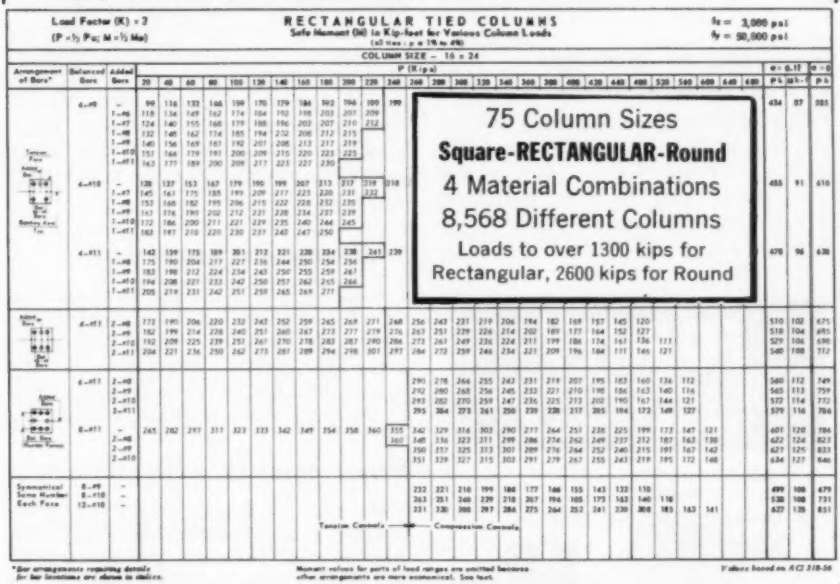
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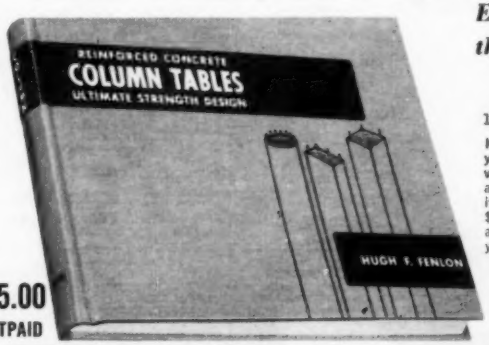

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Freeman to manage Master Builders Seattle office

The Master Builders Co., Cleveland, have announced the appointment of Don W. Freeman as manager of their new branch office in Seattle. Mr. Freeman joined the organization in 1956.

Argue elected president of Stone and Webster

Fred W. Argue was recently elected president of Stone and Webster Engineering Corp., Boston. Mr. Argue joined the firm in 1941 after more than 20 years in the fields of power plant operation and engineering education.

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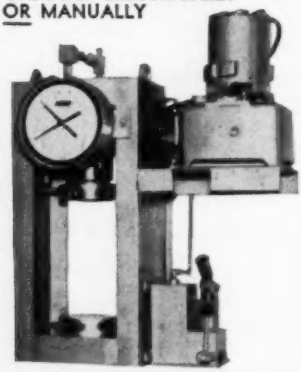

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1960 ASTM Committee Week

Over 1400 engineers and scientists — second largest attendance in the history of the American Society for Testing Materials "Committee Week" — participated in committee work sessions during the week of February 1 in Chicago. In 5 days, 35 of the society's technical committees held 351 meetings of subcommittees and working groups. ASTM Committee C-12, Mortars for Unit Masonry, and Committee C-15, Manufactured Masonry Units, were among the main technical committees which met.

Wolfert joins Quick-Way

Paul J. Wolfert has been appointed director of sales for Quick-Way Truck Shovel Co., Denver. Before joining the firm, Mr. Wolfert was sales manager of Fairfield Engineering Co.'s Standard Products Division at Marion, Ohio.

Mayer assumes AED presidency

Herbert J. Mayer, executive vice-president of Western Machinery Co. and general manager of Edward R. Bacon Co., both in San Francisco, has been named president of Associated Equipment Distributors. His appointment followed the untimely death of Jewel A. Benson, who less than a month earlier had taken the oath of office as the association's president.

At the same time, Richard F. Newlin of Newlin Machinery Corp., Kansas City, Kan., was inducted as AED senior vice-president.

In a special appointment by the board of directors, John R. Borchert of Borchert-Ingersoll, Inc., St. Paul, was named a vice-president. Other 1960 officers of AED include: Braxton Blalock, Jr., Blalock Machinery and Equipment Co., Atlanta, vice-president; R. Boyd Somerville, Ontario Supply and Equipment Ltd., Toronto, Ont., Canada, vice-president; and W. R. Parnell, Construction Machinery Corp., Shreveport, La., treasurer.

McCalley joins KAPL staff

Robert B. McCalley, Jr., has joined the staff of Knolls Atomic Power Laboratory as a consulting engineer in the SAR (Submarine Advanced Reactor) Project.

KAPL, in Schenectady, N.Y., is operated by General Electric Co. for the Atomic Energy Commission. SAR is engaged in the development of long-life reactor cores for the Navy's atomic submarine fleet. The project was originally started to design and develop twin reactors for the huge nuclear submarine, Triton, commissioned last November 10. SAR is still responsible for operation of the prototype reactor, located at KAPL's West Milton site, and for procurement of the second fuel loading for the Triton.

For the past 2 years, Dr. McCalley has been employed by the AEC as a supervisor of a fundamental research program in problems of reactor containment. From 1955 to 1958 he worked in stress and vibration analysis for nuclear reactor components at KAPL.

Engineering group honors Holcomb

Glenn W. Holcomb, head of the department of civil engineering at Oregon State College, Corvallis, was recently chosen "Oregon Engineer of the Year" by the professional engineers of Oregon.

Rand appointed to Form-Crete sales staff

The Florida Division of Food Machinery & Chemical Corp. has announced the appointment of Richard G. Rand as Form-Crete sales engineer for the northeastern territory. Mr. Rand will make his headquarters in Bristol, Conn.

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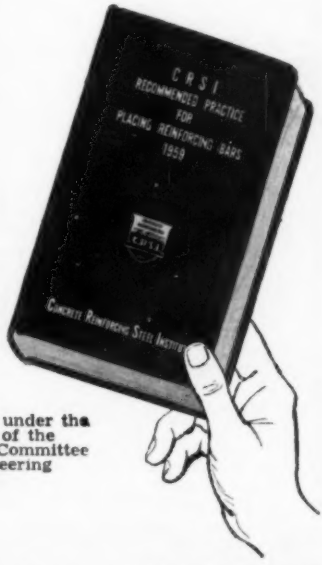

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Honor Roll

January 1 - March 31, 1960

In the past 3 months more than 150 members have added their names to the ACI Honor Roll. Why don't you introduce a new member to the Institute next month and keep our membership trend spiralling?

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Salvador Rodriguez 3
Howard J. Rosenberg ... 3
H. Rusch 3
R. S. Sandhu 3
Herbert A. Sawyer, Jr. .. 3
C. H. Scholer 3
Morris Schupack 3
H. M. Schwartz 3
Harold J. Sexton 3
M. F. A. Siddiqui 3
A. L. Small 3
John D. Smith 3
S. Szalwinski 3
F. K. Taskin 3
J. Antonio Thomen 3
Alex Tobias 3

D. A. Van Horn 3
Ellis S. Vieser 3
Sam Walden 3
Carl Weber 3
R. H. Wildt 3
Cedric Willson 3
J. A. Wineland 3
Mark R. Woodward 3
Milton H. Zara 3
Jayantilal S. Alagia 2
Arthur H. Andersen 2
Charles O. Baird, Jr. 2
John E. Bower 2
D. Campbell-Allen 2
Francisco Castano Hernandez 2
Guillermo Castellanos G.. 2
M. H. Chapman, Jr. 2
Clayton M. Crosier 2
Graham Earle 2
G. Grenier 2
Martin J. Gutzwiller 2
Robert B. Harris 2
Gregorio Hernandez 2
William W. Hotaling, Jr. . 2
Raja A. Iliya 2
Fred W. Jacobs 2
Ralph F. Jones 2
Howard R. May 2
Abel Moreno Przespolewski 2
R. R. Neal 2
Carlos Luis Nebreda 2
Carlos Isunza Ortiz 2
Raoul E. Pallais 2
Gerald F. Paulson 2
Roberto E. Prata Lou ... 2
Luis G. Restrepo S. 2
Walter D. Rudeer 2
Ingvar Schousboe 2
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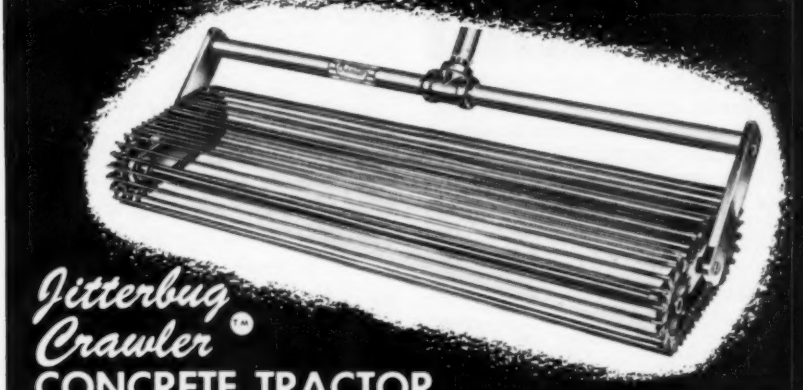
E. C. Sword 2
Rogelio Bonilla Torres ... 2
Leon A. Yacoubian 2
Roger M. Zimmerman ... 2
Antonio Zuniga Ayala ... 2

New Concept for CONCRETE!



Jitterbug Crawler™ CONCRETE TRACTOR

NO OTHER DEVICE EVER DEVELOPED FOR THE PREPARATION OF POURED CONCRETE SLABS COULD EVER MAKE

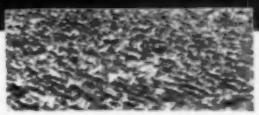
These Proven Claims . . .

- **TRACTORING MAKES WORKABLE SURFACE . . . FAST**
  Longer contact with mix puts coarse aggregate down until "fines" rise around and above.
- **TRACTORING ENDS WADING IN THE MIX**
  Because you "tamp" a large area with an easy push-pull motion, jitterbugging goes many times faster.
- **TRACTORING "LIBERATES" ONE MAN**
  Since it's so easy to use, you can use your jitterbug man for many other jobs besides tamping.
- **TRACTORING MAKES NEW TEXTURES POSSIBLE**
  New textures and new traction surfaces available without costly preparation.

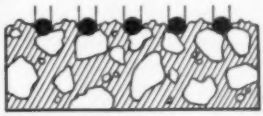
CRAWLING JITTERBUG®

CONCRETE TRACTOR MEANS BETTER CONCRETE . . . IN LESS TIME . . . AT LOWER COST!

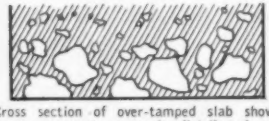
Precision engineered and fully job-proven, the Crawling Jitterbug Concrete Tractor consists of 46 parallel steel rods, each 35" across, operated by gear-driven sprocket chain. Easily operated by one unskilled man, without fluctuation in performance caused by fatigue! Completely assembled, with 12-foot sectional handle, ready to use on the job. Complete, $97.50, FOB Factory in Kansas City. Or write for FREE descriptive booklet on this sensational new machine!



Close-up photo of Tractored surface shows how concrete Tractor's parallel bars press straight down into mix, allowing medium and fine particles to rise around them. No other method leaves surface prepared as shown in this photo!



Cross sectional diagram shows how Tractors rods "ride the rock," putting the larger aggregate just beneath surface, to improve aggregate distribution and help to create improved concrete.



Cross section of over-tamped slab shows coarse aggregate unevenly distributed and too far from surface. Such structure makes for unsatisfactory wear, as well as surface defects.

Cross section of untamped slab shows the coarse aggregate right at surface, waiting to catch blade of finishing trowel. Finished slab with this structure is more likely to spall quickly.

Goldblatt TOOL COMPANY

FREE Booklet describes new method completely—provides facts and specs on this great machine! Write for your copy, today!

1922 Walnut Street **Kansas City 41, Missouri**

New Members

The Board of Direction approved applications in the following categories: 127 Individual, 12 Corporation, 18 Junior, and 22 Student, making a total of 179 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership on Apr. 1, 1960, was 10,253.

INDIVIDUAL

AHMED, MIRZA NIAZ, Lahore, Pakistan (Sr. Draftsman, Batala Engrg. Co., Ltd.)
ALLEN, ALFRED K., Montgomery, Ala. (Vice-Pres. & Mgr., Heavy Constr. Div., Blount Brothers Constr. Co.)
AMAZON, E., Del City, Okla. (Struct. Engr., Capehart Corp., Dynamic Electronics Div.)
ANCIZAR-DUQUE, MANUEL, Bogota, Colombia
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AYRES, GEORGE H., Wallingford, Conn. (Vice-Pres. & Mgr., Wallingford, Conn. Lab & Office)
BAKO, JOHN E., San Francisco, Calif. (Struct. Engr.)
BALLESTEROS, PONFIRIO, Monterrey, N. L., Mexico (Research Prof., Technologico de Monterrey)
BANCIU, V. MIRCEA, Berkley, Mich. (Struct. Engr., Giffels & Rossetti)
BASORA, JOSE L., Rio Piedras, P. R. (Struct. Designer, Reed, Basora & Menendez)
BEATTY, JOHN M., Troy, N. Y. (Prof., Struct. Engrg., Rensselaer Polytechnic Institute)
BEAUCHAMP, J. C., Ottawa, Ont., Canada (Prof. of Strength of Matls., Ottawa Univ.)
BEERMAN, F. RICHARD, Sonora, Calif. (Mgr., Concrete Plant, Tuolumne Transit Mix, Inc.)
BEREZNICKI, JOSEPH STANLEY, Edmonton, Alta., Canada (Proj. Engr., Universal Constr. Co., Ltd.)
BESIER, RUDOLPH F., Denver, Colo. (Assoc., Ketchum, Konkel & Hastings)
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BOZEMAN, ROBERT L., Baton Rouge, La. (Gen. Contr.)
BRANDYS, GEORGE, Don Mills, Ont., Canada (Struct. Engr., Alex Tobias & Assocs., Ltd.)
BROOKS, BOYD S., Coronado, Calif. (Force C.E., Cdr., Naval Air Force, Pacific Fleet)
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CARTER, CARROLL L., Macon, Ga. (Est., Engr., Chris R. Sheridan & Co.)
CASTLE, LEWIS N., Des Moines, Iowa (Constr. Supv., Garmers & Stiles Co.)
CHAN, CHARLES SAI-IP, Dearborn, Mich. (Struct. Engr., Giffels & Rossetti)
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CONWAY, GEORGE O., JR., Philadelphia, Pa. (Conc. Cons. & Constr. Engr., Ambric Testing & Engrg., Assocs., Inc.)
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DESENTIS, ALFONSO, Mexico, D. F., Mexico (Arch., Tecnocreto S. A.)
DOLAR-MANTUANI, LUDMILA M., Toronto, Ont., Canada (Petrographer, Hydro-Electric Power Comm. of Ont.)
DONOVAN, DANIEL K., Bethesda, Md. (Research Engr., Larsen Prods. Corp.)
DUNN, E. S., Greensboro, N. C. (Plt. Mgr., Ready-Mixed Concrete Co.)
ENGELEN, T. A., Springfield, Minn. (Constr. Supv., The Engelen Corp.)
FAVELA LOZOYA, FERNANDO, Teziutlan, Pue., Mexico (Gen. Supt., Apulco Project., Ingenieros Civiles Asociados, S. A. De C. V.)
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HUNT, THEODORE W., Chicago, Ill. (Engr., PCA)
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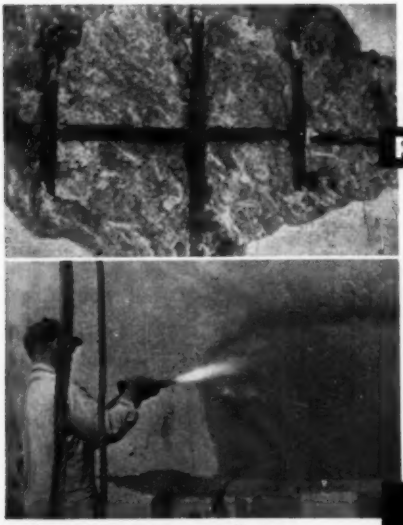

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Spalled concrete generally results from oxidized reinforcing rods. Unless decay is halted, serious structural weakness and huge repair bills will result.

With shotcrete, skilled Western technicians can restore disintegrated concrete structures to original soundness and strength. All work done under contract, fully insured and guaranteed. No materials for sale.

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WU, TZE SUN, Detroit, Mich. (Assoc. Prof. of Civil Engrg.)
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MUSSELMAN, ALAN M., Warminster, Pa. (Engrg. Cons., Insurance Co. of North America)
SANDOVAL L., HERNAN, Cali, Colombia (Water Works Mgr., Empresas Municipales de Cali)
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VARGAS-CESANI, CARLOS, Mayaguez, P. R. (Constr. Engr., George F. Turner, Res. Engrs. Off.)

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ANNONI, FRANCISCO, Mayaguez, P. R. (College of Agric. & Mech. Arts)
BARRERA, ANTONIO, Bogota, Colombia
CAMPBELL, CARROLL E., Chico, Calif. (Chico State College)
CUERONI, ANDREW P., Framingham, Mass. (Worcester Polytechnic Inst.)
ELZINGA, PAUL, Holland, Mich.
FERNANDEZ E., RAFAEL, Guatemala, Guatemala
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HOHMANN, JOSEPH C., JR., West Roxbury, Mass. (Northeastern Univ.)
IREN, ERGUN, Lawrence, Kansas (Univ. of Kansas)
JUAREZ L., ADRIAN, Guatemala, Guatemala (Univ. de Sn Carlos)
LEAUMONT, WALTER J., JR., Baton Rouge, La. (L. S. U.)
MELENDEZ FLORES, JOSE MANUEL, Santurce, P. R. (College of Agric. & Mech. Arts)
NOOR, AHMED KHAIRY, Champaign, Ill. (Univ. of Ill.)
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SARAJOTI, AMPHORN, Worcester, Mass. (Worcester Polytechnic Inst.)
SIMPSER, BORIS, Mexico, D. F., Mexico (Univ. Nacional Autonoma de Mexico)
SOLIMAN, FAROUK, New York, N. Y. (Colombia Univ.)

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Caulking sealant

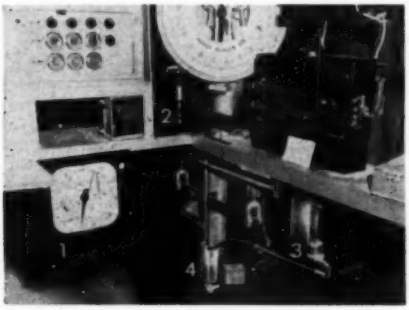
A sealant developed by Grayguard is based on Du Pont's Hypalon synthetic rubber. The mastic sealant is said to bond tightly to all materials and self-cures to a resilient rubber seal. Developer claims that field trials have shown the sealant to be applicable for construction and expansion joints in concrete or for tight closure where metals are joined to masonry. Retention of whiteness and color is one of the attributes cited by the company.

Features of the material cited by the manufacturer are the fact that it doesn't slump or sag, even in wide or deep vertical joints; it

works well in hot or cold weather; becomes tack-free in about 12 hr.—Grayguard, Inc., P.O. Box 1644, Wilmington 99, Del.

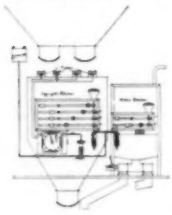
Automatic moisture control

The Moisture Matic compensator system indicates, records, and compensates for variations in moisture content of both fine and coarse aggregates by making up a liquid counterweight which goes on the scale while the aggregates are being weighed. Variations in moisture content are converted to a parallel variation in air output which in turn maintains a flow of water that is directly proportional to the moisture in the flowing



aggregates. Photo shows: (1) and (2)—controller and valve which make up liquid counterweight; (3) and (4)—plastic counterweight containers attached to aggregate and water scales.

Line drawing shows how the unit works: Rugged probes are mounted on and insulated from the aggregate weight batcher in such a position that they sense the moisture in the aggregates while they are falling from the bin into the weigh batcher. The meter instantaneously converts changes in the moisture content of the aggregates to a directly proportional change in liquid counterweight flow; it also indicates and records the moisture content, acting as the nerve center for the entire system. Control valve governs the flow of water and changes on signal from meter. Liquid counterweight is discharged automatically while batch is being discharged. —Concrete Controls Corp., 1012 E. Geneva Road, Wheaton, Ill.



Plasticizer for use in block manufacture

NDK Plasticizer No. 4 has recently been introduced for use in molding concrete block and other shapes. The product is used in conjunction with aggregate at time of mixing, making block virtually waterproof, says the manufacturer.

NDK Laboratories further state that use of Plasticizer No. 4 gives better binding qualities to the aggregate thus avoiding sagging and distortion of shapes; also prevents hairline cracks and crumbling corners; makes products somewhat lighter in color and gives a smoother finish. Sold in concentrated form in 55-gal. steel drums.—NDK Laboratories, 6912 West Cedar St., Milwaukee 13, Wis.

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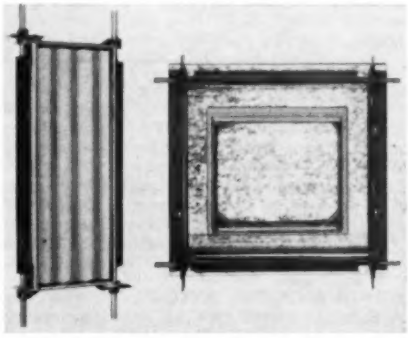
Silo stave form

A special plastering lip has been incorporated in a corrugated silo stave form developed by O K Manufacturing Co. for use with the wet casting method. The lip runs entirely around the inside edge of the form and makes an indentation in the finished stave about 1/8 in. deep and approximately 3/8 in. wide. When the staves are placed in position the result is a groove about 3/4 in. wide at each vertical joint and about 1/4 in. wide at



each horizontal joint. This is an aid to faster, easier plastering and a better seal, the manufacturer states. Size of the finished stave is 11 x 30 x 2 3/4 in.

On the silo door form the lip runs around the outside edge. Size of the finished door section is 30 x 30 x 2 3/4 in. Both sections of the door form are prepared for mounting on 3/4 in. plywood.—O K Manufacturing Co., Inc., 2824 22nd Ave., South, Minneapolis 7, Minn.

Concrete forming system

The Cam-Lock system, a newly patented concrete forming system, has been developed by Gates & Sons, Inc. The system embodies the use of flexible, inexpensive, forming materials, S4S 2x4's or 2x6's with 4x8-ft plywood sheets. No ribbing or special hardware attached to the panels is necessary and the use of stiff backs and walers is actually cut in half states the manufacturer. The Cam-Lock tie allows the system to be used with either loop-end or button-type ties through either holes or slots drilled in the panels; walers may be used either vertically or horizontally. The Cam-Lock bracket holds the 2x4 waler in place by locking to either the loop-end or button-type tie through camming pressure. Further rigidity of the form may be obtained through use of the stiff-back cam which is a camming arm that connects to ears on the back of the Cam-Lock bracket locking either a 2x4 or 2x6 in place. —Gates & Sons, Inc., 80 S. Galapago St., Denver 23, Colo.

Literature Available

Pertinent details on the latest equipment and products on the market are available in recently released literature. Exact titles of the booklets and catalogs are indicated in capital letters. They may be requested directly from the manufacturers listed below.

RECRUITING PRACTICES AND PROCEDURES—A universally applicable guide for the recruiting of engineering college graduates has been published by the American Society for Engineering Education. The 8-page leaflet lists in detail recommended practices and procedures by employers, colleges, and students. Price is $0.10 per copy; $3 per 100. —American Society for Engineering Education, University of Illinois, Urbana, Ill.

EUROPEAN TECHNICAL DIGESTS—A monthly compilation of technical innovations appearing in the technical press all over Europe. Each month over 1000 technical journals appearing in 13 different countries and in 11 different languages are searched for information on new processes, methods, apparatus, or materials which could be readily employed in industry. The magazine is published by the Agence Européenne de Productivité, an inter-government body within the Organisation Européenne de Coopération Economique to promote the advancement of management education, the spread of the latest technical skills and management techniques, and generally to facilitate an exchange of information. Yearly subscription, $12. — Agence Européenne de Productivité, Organisation Européenne de Coopération Economique, 2 Rue Andre-Pascal, Paris.

POWDER-CUTTING APPLIED TO CONCRETE—Describes the powder-cutting process in which iron particles, or iron powder as it has generally become called, are fed into an oxygen-cutting flame. Describes recent apparatus developments and new operating techniques which now make it practical to powder-cut concrete in a wide variety of applications—Linde Co., Division of Union Carbide Corp., 30 East 42nd St., New York 17, N.Y.

WRITING AND PUBLISHING YOUR TECHNICAL BOOK—"Anyone who can write a good business letter has all the literary ability he needs to write a good technical or professional book." So begins this 50-page manual published by F. W. Dodge Corp. The booklet is written to help authors organize and develop their ideas for books. It answers a great many questions about the author-publisher relationship and it also presents many practical checklists for preparing material for publication.—Dodge Books, F. W. Dodge Corp., 119 W. 40 St., New York 18, N.Y.

PLUMBING FOR FLEXICORE DECKS—Booklet represents results of study with architects, engineers, and contractors to perfect efficient methods of providing openings in Flexicore decks for plumbing and heating lines. Describes typical plumbing conditions, showing efficient methods of providing openings for each of these conditions. Includes details covering most typical conditions the plumber will encounter.—The Flexicore Co., Inc., 1932 East Monument Ave., Dayton 1, Ohio.

VERMICULITE FIRE-RESISTANCE RATINGS—Gives fire-resistance ratings on vermiculite plaster, acoustical plastic, concrete, and Type-MK fireproofing product. Lists latest fire tests including several made on Type MK fireproofing which can be applied directly to galvanized metal and steel beams in one coat.—Vermiculite Institute, 208 South LaSalle St., Chicago 4, Ill.

SCREED ATTACHMENT FOR PAVING SPREADERS (Bulletin SD-125)—Describes a screed attachment for concrete paving spreaders that eliminates the need for an extra finisher. Bulletin details one particular job on which the Model 5M screed attachment was used.—Sales Promotion Department, Blaw-Knox Co., Construction Equipment Division, Mattoon, Ill.

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. All who have an interest in concrete are eligible for membership. The grades of membership are described below.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI Journal provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings, they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

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The undersigned hereby applies for admission to the American Concrete Institute as ☐ *Individual* ☐ *Corporation* ☐ *Contributing* ☐ *Junior* ☐ *Student Member and agrees to be governed by the Charter and Bylaws.*

Name and complete mail address of proposed membership (Address to which Journal is to be mailed—please letter) _____

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For Corporation Membership, ACI representative will be _____

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For our records, please complete both sides of application.

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